JOURNAL

OF THE

AMERICAN WATER WORKS ASSOCIATION

The Association is not responsible, as a body, for the facts and opinions advanced in any of the papers or discussions published in its proceedings.

VOL. 1 MARCH 1914 No. 1

FOREWORD

With this issue, the American Water Works Association enters upon a new era of advancement under its new Constitution.

The establishment of sections in various parts of the country, now forming or already formed, requires a more prompt medium for the early production of our proceedings and papers. Realizing the importance of the matter, the Executive Committee of the American Water Works Association has authorized the issuance of a quarterly publication, of which this issue is the first number.

It is planned to have the first number issued early in the year prior to the Convention, and contain papers which will later be discussed at the Convention.

A second number will contain proceedings of the Convention and such discussions as may be available for publication.

The third and fourth numbers will complete the discussions, publish additional papers submitted to the sections, make necessary announcements of local meetings, and contain revised lists of membership.

It is hoped by this procedure to present more promptly any papers of merit, without the delays attendant upon a single annual issue, and furnish a repository for water works literature and discussion, which will be in closer touch with the needs and interests of the membership, beside being a greater incentive to contributors.

This result will be only attained by the hearty support of the Association, and larger efforts by the Editor and Publication Committee, but the Committee and the Editor enter cheerfully upon the task, believing that a successful Quarterly Journal will result in greater interest and increased prosperity to the Association, as a whole.

The recently formed New York Section is a fair index of the needs of the Association in enlarging its scope and forming opportunities for local interest, not apart from the parent organization but supplying opportunity for local fellowship, interest, and discussion. Other State Associations are considering joining the American Association, and will doubtless in time come to enjoy the larger privileges and opportunities that the national organization can give them, especially in providing a common repository for papers, discussions, and notices.

Let us hope that the new departure will prove that the American Water Works Association will be a greater benefit than ever before to its members.

JOHN W. ALVORD,
Chairman, Publication Committee.

AFFAIRS OF THE ASSOCIATION

CHANGES IN MEMBERSHIP

The following named have applied for membership and been duly elected by the membership committee:

ACTIVE

	Elected
G. A. Solomon, Cons. Engr., Atlanta, Ga.	October 22, 1913
D. A. Reed, Mgr. Water & Lt. Dept., Duluth, Minn.	October 22, 1913
Leo Hudson, Cons. Engr., Pittsburgh, Pa.	October 22, 1913
L. I. Birdsall, Minneapolis, Minn.	October 22, 1913
Patrick Gear, Supt. Water, Holyoke, Mass.	November 11, 1913
John Gaub, Filtration Plant, Washington, D. C.	November 11, 1913
James Ewing, Asst. Engr. Water Dept., Newcastle, N.S.W.	November 11, 1913
J. J. Ferris, Const. Engr., Jersey City, N. J.	December 11, 1913
C. M. Whitney, San Francisco, Cal.	December 11, 1913
J. F. Mangold, C.E., Grinnell College, Grinnell, Iowa	December 11, 1913
Guy Britton, C.E., J. C. Water Dept., Boonton, N. J.	December 22, 1913
F. J. Longley, Chemist, Western N. Y. Water Co., Niagara Falls, N. Y.	December 22, 1913
F. H. Eno, C.E., Ohio State University, Columbus, O.	December 22, 1913
F. J. Glueck, Mountain Water Supply, Philadelphia, Pa.	December 22, 1913
C. E. Titus, Northfield Land & Water Co., Colorado Spr., Col	December 22, 1913
David Davis, Mayor, Litchfield, Ill.	December 22, 1913
T. R. Duggan, Ph.D., Chemist, New York, N. Y.	December 22, 1913
F. A. Dallyn, C.E., Provincial San. Engr., Toronto, Ont.	January 17, 1914
E. H. Mark, M.D., State San. Engr., Bowling Green, Ky.	January 17, 1914
F. deP. Hone, C.E., New York, N. Y.	January 17, 1914
W. H. Gould, San. Engr., Toledo, O.	January 17, 1914
C. E. Chester, C.E., City Water Co., Shelbyville, Ill.	January 17, 1914
B. L. Williams, Cons. Engr., East Orange, N. J.	January 17, 1914
G. S. Cheny, Jr., C.E., Philadelphia, Pa.	January 17, 1914
Guy Eldredge, Chemist, Filtration Plant, Fort Worth, Texas	January 17, 1914
W. M. Cobleigh, Prof. Chemistry, State College, Bozeman, Mont	January 17, 1914
W. E. Everette, Cons. Technologist, Tacoma, Wash.	January 17, 1914
G. L. Bean, Hydraulic Engineer, Philadelphia, Pa.	January 21, 1914
L. H. VanBuskirk, Asst. Engr., State Board of Health, Columbus, O.	February 9, 1914
H. W. Taylor, Hyd. & San. Engr., Albany, N. Y.	February 9, 1914
R. S. Buzzell, Chemist Supt. Filtration, Flint, Mich.	February 9, 1914

Elected

C. C. Young, Director State Water Survey, Lawrence, Kan. February 9, 1914
J. R. McClintock, 170 Broadway, New York, N. Y. February 9, 1914
L. A. Fisher, Concord, N. C. February 9, 1914
A. E. Walradt, 26 Liberty St., New York, N. Y. February 9, 1914
H. A. vonSchon, Cons. Hyd. Engr., Detroit, Mich. February 9, 1914
J. M. Hayden, Dept. Water, Gas & Electricity, Brooklyn N. Y. February 9, 1914
H. P. Stearns, Queens County Water Co., Far Rockaway, N. Y. February 9, 1914
S. H. Durland, Queens County Water Co., Far Rockaway, N. Y. February 9, 1914
C. H. Fisher, Fire Protection Engr., Jersey City, N. J. February 9, 1914
George Hogarth, Asst. Engr., Dept. Public Works, Toronto, Ont. March 10, 1914
A. Lederer, M.D., Sanitary Dist. of Chicago, Chicago, Ill. March 10, 1914
F. D. West, Chemist, Torresdale Laboratory, Philadelphia, Pa. March 10, 1914
G. L. Watters, Hyd. Engr., South Bethlehem, Pa. March 10, 1914
F. L. Waterman, Supt. Water, Endicott, N. Y. March 10, 1914
G. C. Pruett, City Engr., Supt. Water, Miles City, Mont. March 10, 1914
A. J. Walker, Chief Engr. Water Dept., Ambler, Pa. March 10, 1914
W. M. Scott, Cons. Engr. Winnipeg, Manitoba. March 10, 1914
R. I. Randolph, 1870 West Adams St., Chicago, Ill. March 10, 1914
R. B. Champion, Supt. Public Works, Holland, Mich. March 10, 1914

CORPORATE

Galt Water Co., Galt, Ont. October 22, 1913

ASSOCIATE

NcNab & Harlin Mfg. Co., 55 John St., New York, N. Y. October 22, 1913
Reisert Aut. Water Purif Co., 30 Church St., New York, N. Y. December 11, 1913
American Bitumastic Enamels Co., 109 Broad St., New York, N. Y. December 22, 1913
Arnold, Hoffman & Co., Philadelphia, Pa. December 22, 1913

DEATHS SINCE LAST CONVENTION

Jay Chatham, Supt, Water Works, Iowa City, Ia. September 29, 1913
Arthur L. Adams, Cons. & Hyd. Engr., San Francisco, Cal. October, 1913
Alba M. Holmes, Hyd. Engr., Grand Rapids, Mich. November, 1913
Allen T. Prentice, Mech. Engr., Chicago, Ill. December, 1913
Murray Forbes, Supt. Irwin Water Co., Greensburg, Pa. December, 1913
A. B. Tracey, Engr. Consolidated Water Co., Utica, N. Y. December, 1913
O. A. Madison, Supt. Water Works, Charlotte, N. Y. December, 1913
John C. Kelley, Director Water Works, New York, N. Y. January, 1914

	Elected
T. Kobayashi, Chf. Engr. Water Works, Osaka, Japan	January, 1914
Stuart Wood, Prest., 400 Chestnut St., Philadelphia, Pa	March, 1914
E. L. Carroll, Jacksonville, Fla	March, 1914

RESIGNED SINCE LAST CONVENTION

F. H. Bass, Minn. State Board of Health	Minneapolis, Minn.
M. W. Kingsley, C.E.	Cleveland, O.
Edward B. Burch, Cons. Engr	Minneapolis, Minn.
A. K. Hale, Treas. & Mgr. Water Co	Ann Arbor, Mich.
Edmund Mather, Prest. Water Board	Harrisburg, Pa.
Kenneth Torrence	Schenectady, N. Y.
Philip S. Wolle, Bacteriologist	Johnstown, Pa.
Jenkins Brothers	71 John St.

Members as per last report:		
Honorary	7	
Active	835	
Corporate	58	
Associate	118	1018
Elected:		
Active	50	
Corporate	1	
Associate	4	55
		1073
Losses:		
Died	11	
Resigned, Active	7	
Associate	1	19
Members on roll March 27, 1914		1054
Applications in hands of Membership Committee (not yet acted upon):		13
		1067

The Corporate Active have, in accordance with the new constitution, been changed to Corporate and Associate. Water departments and water companies have been changed to Corporate list, so that the new membership list, with the new members now stands:

Honorary	7	
Active	869	
Corporate	65	
Associate	113	1054

PHILADELPHIA CONVENTION

The preparations for the Philadelphia Convention, May 11–16, 1914, are nearly completed. The local arrangements are in the hands of the Local Committee of Arrangements.

COMMITTEE ON ARRANGEMENTS

MR. CARLTON E. DAVIS, *Chairman*

MR. W. H. BOARDMAN — MR. EDWARD J. CATTELL
MR. J. W. LEDOUX — MR. A. W. MACCALLUM
MR. C. W. SUMMERFIELD — MR. H. G. H. TARR
MR. JOHN C. TRAUTWINE, JR.

RECEPTION COMMITTEE

HONORABLE RUDOLPH BLANKENBURG, Mayor, *Chairman*

MR. GEORGE F. BAYER.....President Philadelphia & Reading Ry.
MR. SAMUEL T. BODINE.....President United Gas Improvement Co.
REV. RUSSELL H. CONWELL.....2020 N. Broad St.
MR. NORRIS L. COOKE.....Director Dept. of Public Works.
MR. NATHAN T. FOLWELL.....2008 Spring Garden St.
MR. ALBA B. JOHNSON.....President Baldwin Locomotive Works
MR. T. E. MITTEN.....Philadelphia Rapid Transit Co.
MR. SAMUEL REA.....President Pennsylvania R. R. Co.
DR. EDGAR FAHS SMITH.....Provost Univ. of Pennsylvania
MR. WILLIAM T. TILDEN.....250–54 N. Front St.

The Water Works Manufacturers Association have appointed the following committees:

ENTERTAINMENT COMMITTEE

MR. C. R. WOOD, *Chairman*

MR. N. E. BARTLETT — MR. JAMES H. CALDWELL
MR. I. S. HOLBROOK — MR. W. H. VANWINKLE, SR.

LADIES ENTERTAINMENT COMMITTEE

MRS. N. E. BARTLETT — MRS. GEORGE M. COSTELLO
MRS. CARLETON E. DAVIS — MRS. J. W. LEDOUX
MRS. C. W. SUMMERFIELD — MRS. SETH M. VANLOAN

TRANSPORTATION COMMITTEE

MR. F. W. SHEPPERD, *Chairman*

MR. T. C. CLIFFORD — MR. H. M. LOFTON
MR. F. J. BRADLEY

EXHIBITION COMMITTEE

MR. W. H. VANWINKLE, *Chairman*

MR. T. D. FAULKS — MR. V. E. ARNOLD
MR. F. W. SLEEP — MR. G. M. KEEFER

The program of papers will be unusually interesting; nine of them are printed in this journal, and are open for discussion at the convention, or by letter. The fullest discussion of all of the papers is earnestly requested. There are other equally good papers on the program, some of which will be ready in pamphlet form for distribution before the convention.

Thursday will be Superintendents' Day, the question box will be opened and a general exchange of experiences entered into. A few short practical papers by water works superintendents will be presented. A partial list of the questions has been sent out, and a full list will be printed and sent to all members before the convention. Send early to the Secretary of the Association any questions you desire answered; also any topic you desire to have discussed in the experience meeting.

Everything points to a large attendance. Make your arrangements to attend. The Headquarters will be at the Bellevue-Stratford, Philadelphia's leading hotel. Other hotels near by also offer good accommodations. Full announcement will be made soon.

TRANSPORTATION

No reduced rates will be granted for this convention. Regular excursion rates will probably be in force. Ask your local ticket agent about it. If no rate is in force for Philadelphia, Atlantic City rates can be obtained.

THE SECTIONS

A section was formed by the members residing in the States of New York and New Jersey, to be called the New York Section. This section has held two meetings, the minutes of which follow:

MINUTES OF FIRST MEETING OF NEW YORK SECTION

A meeting was held at the Manhattan Hotel January 20, 1914, attended by 44 active members of the association and 41 associate members and guests.

Mr. Allen Hazen was chosen temporary chairman.

Mr. Robert J. Thomas, President, and Mr. J. M. Diven, Secretary, of the American Water Works Association, were present, and explained the provisions of the new constitution relating to the formation of local sections.

It was then voted to form a section, and the following constitution, after discussion, was adopted, to be submitted to the Executive Committee of the American Water Works Association, for approval:

CONSTITUTION OF THE NEW YORK SECTION

SECTION 1. The business of the New York Section shall be administered by a Committee composed of five active members of the section. The five members first chosen shall serve one, two, three, four and five years respectively as may be determined by lot. Thereafter one member shall be chosen at the first meeting of the Section held in each Calendar year, and shall serve for five years and until his successor is chosen.

SECTION 2. The Committee shall choose a chairman, a secretary and other officers from its own number, shall fill vacancies, shall appoint sub-committees, as it may deem proper, and shall have full authority to manage the business of the section. Any member of the New York Section may be appointed a member of sub-committees.

SECTION 3. The Committee shall report to the section from time to time any changes or additions to this constitution which it may deem desirable for more efficient management, the same to become operative when approved by a two-thirds vote of the members in attendance at a meeting of the section. The Committee shall submit to the section any proposed change in the constitution offered by ten members, with its recommendations thereon.

SECTION 4. The Constitution of the American Water Works Association, and all by-laws or rules under it, shall apply as far as applicable to the business of the New York Section.

A nominating Committee being authorized, was appointed by the chair, as follows: C. R. Bettes, D. W. French, J. H. Cook, J. M. Diven, J. W. Ackerman.

Mr. George W. Fuller then presented a paper on "The Croton Water, Its Quality and Improvement," and this was discussed.

The Nominating Committee recommended for members of the Governing Committee: Allen Hazen, J. Waldo Smith, Robert E. Milligan, Morris R. Sherrerd, Henry deF. Baldwin.

These nominees were thereupon elected unanimously. The meeting then adjourned.

MINUTES OF SECOND MEETING OF NEW YORK SECTION

The second meeting of the New York Section of the American Water Works Association was held at Hotel Manhattan March 10, 1914. There were present 49 active members, 37 associate members and guests.

The Minutes of the preceding meetings were ordered printed, and the Secretary reported the result of the vote as to the manner of holding the Section meetings. The results were as follows:

For the present arrangement, with lunch	53
For an evening meeting without food or hired hall	14
Did not vote	27
Total returns	94

The President then introduced the first speaker, Mr. George A. Johnson, who presented an abstract of his paper on "Present Water Purification Practice." The paper was discussed by Messrs. F. C. Dunlap, Francis D. West and Joseph P. Siddons, of Philadelphia, also by Messrs. John A. Kienle, Allen Hazen, Wm. P. Mason and Frank E. Hale. The President then suggested that the rest of the discussion could be continued by correspondence, and called upon Mr. Herman Rosenstretter, who gave an illustrated talk on the Newark Water Works. The slides showed the general plan of the supply works and pictures of the reservoirs. Mr. Rosenstreter spoke of the city's policy in acquiring land on the catchment area, of which the city now owns about 50 per cent. He also spoke of the relation of the rain-fall to the runoff, and also of the supply line to the city, its reduction in carrying capacity with use, and the means taken to guard the safe conveying of the water from the reservoirs to the city.

There was no discussion of this paper, and the President declared the meeting adjourned.

INVESTIGATION INTO THE ADVISABILITY OF SUBSTITUTING AGAR FOR GELATINE AS A MEDIUM FOR THE DETERMINATION OF BACTERIAL COUNTS IN WATER ANALYSIS

By W. U. C. Baton

In the revision of the standard methods of water analysis, the committee of the American Public Health Association which had this work in charge made a most radical recommendation in regard to bacterial counts. This revision was published in 1912 and the following is quoted from this edition, pages 77 and 78:

> In the present state of bacteriology there is no method known by which the absolute number of living bacteria in a sample of water can be determined, and all quantitative determinations of bacteria are necessarily of a relative character. This being the case, strict adherence to a standard procedure is of especial importance. . . .
>
> Quantitative bacterial determinations are of especial value as affording the best index of the efficiency of filtration. . . . .
>
> Since gelatine does not give the total number of bacteria in the water, the committee has thought it wise to use Agar incubated at 37° C. as a standard medium. This admits of counts in one day instead of two and give results on the kind of bacteria growing at blood temperature and therefore nearly related to pathogenic types.
>
> The standard medium for determining the number of bacteria in water shall be nutrient agar. All variations from this shall be considered special media.

There can be no doubt as to the stand of the committee. By this report gelatine is absolutely discarded for use in determining the number of bacteria in water. The writer was completely astounded by this extremely radical change. It evidently, judging from adverse expressions which have come to me, affected many others in a similar manner.

A protest has been made against this in a vigorous manner by various men interested. Whipple reports (*American Journal of Public Health* 1913, Vol. 3, No. 1) that a letter of inquiry to twenty leading filter operators brought eighteen replies. A large majority of these either preferred the gelatine or withheld a statement of opinion until they had given the two media a trial side by side.

It is consoling that the laboratory section of the American Public Health Association passed a resolution at the 1912 meeting advising the use of both gelatine and agar and encouraged the gathering of data on which to base a conservative opinion before making so radical a move as proposed by the committee.

As soon as the published report of the committee came to the writer's attention, he at once began to plan the acquiring of data either to justify or disprove the wisdom of the change. He could not conscientiously accept it without a local investigation. The results obtained will prove the wisdom of this apparent distrust and perhaps ultra conservatism.

The writer's previous experiences with agar for the determination of the number of bacteria in water had not been very encouraging. Previous to this time he had always worked with neutral 1.5 per cent agar with or without sugar and litmus. The agar recommended by the committee was different and therein was a possibility that perhaps his adverse opinions were ill conceived.

The work of investigation was outlined as follows:

Four of our routine sampling points were selected for the comparative tests. These represent the waters we have under control. These waters are designated in the tabulations as Class A water, Class B water, Class C water and Class D water.

The frequency of positive tests for coli, also shown in Tables 1 to 4, inclusive, will give an idea of the relative sanitary character of the waters.

The gelatine medium used was made in accordance with standard procedure. The percentage of gelatine was corrected for moisture. The agar medium used was that recommended in the 1912 edition of Standard Methods. This was made up on the basis of dry agar. The gelatine plates were incubated at 20° Centigrade for forty-eight hours. The agar plates were incubated at 37.5° C. for twenty-four hours. Porous covers were used on all agar plates.

The work was begun in October, 1912, and has been continued to the present time. In all some 1898 samples are tabulated. The data submitted extends over a period of sixteen months to February 1, 1914.

Tabulations 1 to 4, inclusive, show monthly averages for the various classes of water previously enumerated. These averages have been further distributed by averaging together only those samples having the same result on the Colon test. In each of these tables, each successive column is indicated as a water of a lower sanitary grade.

TABLE 1

Class "A" water.—Comparison of gelatine and agar counts for various coli conditions

	NEGATIVE 1 cc.			POSITIVE 1 cc.			POSITIVE $\frac{1}{10}$ cc.			POSITIVE $\frac{1}{50}$ cc.			POSITIVE $\frac{1}{100}$ cc.			POSITIVE $\frac{1}{1000}$ cc.		
	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar
1912																		
October				1	135,000	25,500	5	53,200	15,940	4	75,750	13,400	5	72,000	26,200	9	54,111	14,467
November				3	9,100	300	12	9,533	2,233	5	11,680	1,450	6	15,217	1,867	1	17,500	1,800
December				7	13,514	574	9	6,722	487	5	10,200	422	4	18,125	773	5	31,240	425
1913																		
January	5	6,920	405	8	9,088	503	2	31,250	1,125	5	22,200	1,144	6	17,500	979	5	18,720	1,200
February	3	8,226	803	7	11,929	666	6	20,267	413	6	60,733	1,093	3	14,733	640	3	35,000	617
March				2	11,150	248	10	15,190	485	6	10,950	456	6	14,600	577	5	24,517	1,137
April				4	98,750	13,275	7	132,286	23,300	8	23,813	1,553	9	20,722	1,917	2	51,000	3,850
May				2	105,000	27,750	9	175,278	55,744	6	184,250	54,000	3	207,667	18,467	10	125,650	20,360
June							8	116,750	30,125	9	51,278	26,700	7	40,657	17,471	5	79,600	49,880
July							1	16,500	6,600	6	27,667	16,600	11	46,636	19,709	13	61,846	37,769
August							4	27,250	21,000	4	18,250	12,025	3	49,333	24,666	18	26,006	21,250
September				2	23,700	4,700	3	23,000	16,067	5	38,000	38,800	5	41,200	25,600	9	48,111	37,167
October	1	350	475	2	17,800	3,800	4	8,850	5,975	3	27,800	9,400	9	30,556	7,633	11	38,000	17,709
November				3	15,100	2,517	6	16,050	675	7	36,927	1,536	8	32,475	1,156	6	37,083	750
December	1	4,400	180	7	5,757	187	8	10,113	209	10	4,450	131	4	8,650	69	1	5,700	90
1914																		
January	4	556	13	8	293	260	8	2,581	201	5	4,650	398	4	6,725	131	0	.....	.....

TABLE 2

Class "B" water.—Comparison of gelatine and agar counts for various coli conditions

	NEGATIVE 1 CC.			POSITIVE 1 CC.			POSITIVE $\frac{1}{10}$ CC.			POSITIVE $\frac{1}{50}$ CC.			POSITIVE $\frac{1}{100}$ CC.			POSITIVE $\frac{1}{1000}$ CC.		
	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar
1912																		
October				1	18,000	14,000	2	55,250	11,700	4	47,625	8,650	4	117,250	18,700	3	186,000	12,600
November	1	21,000	250	3	26,666	1,066	5	26,100	1,250	3	33,333	1,016	9	32,522	2,522	3	78,666	1,933
December				5	15,520	276	5	8,680	400	8	22,638	450	2	32,000	370	3	36,166	1,150
1913																		
January	8	8,350	271	11	18,973	414	6	19,767	683	15	17,800	556	8	30,063	806	4	14,525	644
February	5	28,000	220	14	103,136	700	13	118,077	750	11	47,500	751	4	32,125	838			
March	3	22,833	272	11	30,455	335	17	15,600	484	8	14,225	685	12	28,208	526			
April				12	83,375	7,283	19	57,316	3,151	10	109,000	12,788	6	44,750	2,173	4	53,500	1,256
May				12	61,125	20,058	19	59,789	11,621	12	48,125	8,083	5	59,500	11,400	2	37,000	10,400
June	1	12,000	22,000	11	37,682	19,155	21	29,690	14,190	11	65,509	14,955	6	44,750	19,850			
July							9	25,722	21,466	19	14,884	10,037	12	13,886	8,233	12	31,217	17,800
August				2	7,250	6,400	9	10,422	13,644	7	9,386	12,143	14	13,779	10,279	7	15,614	17,557
September	4	2,122	840	3	14,083	2,840	3	40,000	31,566	5	28,600	24,180	6	27,583	20,233	4	22,500	24,250
October	2	1,800	245	2	26,250	5,150	4	15,925	3,750	7	25,500	6,614	2	14,400	7,750	10	26,850	14,220
November				6	25,833	618	5	18,700	1,302	8	29,875	1,088	3	53,333	1,508	2	33,250	875
December	1	1,800	36	9	10,111	156	17	11,641	183	6	3,817	164	6	18,750	119			
1914																		
January	11	1,190	19	11	2,543	33	12	5,363	145	9	4,366	228	2	5,300	299	2	2,950	588

TABLE 3

Class C water.—Comparison of gelatine and agar counts for various Coli conditions

	NEGATIVE 10 cc.			POSITIVE 10 cc.			POSITIVE 1 cc.			POSITIVE $\frac{1}{10}$ cc.		
	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar
1912												
October	3	643	127	11	465	113	7	741	132			
November	1	4,500	180	13	1,394	59	7	964	58	2	10,250	11
December	2	2,388	21	12	1,514	51	5	3,960	80	4	2,125	38
1913												
January	7	236	32	8	1,461	91	10	1,335	50	1	1,800	55
February	6	5,892	34	16	5,800	41	2	13,500	55			
March	4	75	19	14	319	28	7	803	23	1	750	17
April	10	251	43	11	217	20	2	320	192	1	250	3
May	16	447	23	9	436	76						
June	16	215	17	9	174	24						
July	3	105	27	19	92	34	3	167	35			
August	13	28	18	9	42	34	1	25	28			
September	12	38	55	10	116	78	3	67	56			
October	4	65	16	11	109	28	11	368	77	1	170	11
November	1	900	27	13	1,175	35	8	3,275	28	2	5,350	38
December	5	1,452	5	16	970	11	3	1,367	4			
1914												
January	10	3,275	8	13	1,128	10	2	2,050	2	1	190	3

TABLE 4

Class D water.—Comparison of gelatine and agar counts for various Coli conditions

	NEGATIVE 10 cc.			POSITIVE 10 cc.			POSITIVE 1 cc.			POSITIVE $\frac{1}{10}$ cc.		
	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar
1912												
October	17	25	18	6	357	53	1	155	34	.....	.....	.....
November	18	153	20	8	2,023	26	1	190	25	.....	.....	.....
December	18	60	11	8	1,625	27	4	565	13	.....	.....	.....
1913												
January	22	50	16	8	697	42	.....	.....	.....	.....	.....	.....
February	20	51	10	3	215	10	1	240	20	.....	.....	.....
March	22	22	8	4	135	43	.....	.....	.....	.....	.....	.....
April	24	11	8	5	62	22	.....	.....	.....	.....	.....	.....
May	27	16	8	2	56	15	.....	.....	.....	.....	.....	.....
June	27	12	6	1	45	4	.....	.....	.....	.....	.....	.....
July	17	14	9	10	37	14	.....	.....	.....	.....	.....	.....
August	24	8	6	3	7	38	.....	.....	.....	.....	.....	.....
September	22	13	10	5	67	56	.....	.....	.....	.....	.....	.....
October	21	3	4	4	27	46	2	312	14	.....	.....	.....
November	19	41	4	3	733	7	2	2,200	10	.....	.....	.....
December	28	83	1	2	165	7	.....	.....	.....	.....	.....	.....
1914												
January	26	144	2	3	307	9	.....	.....	.....	.....	.....	.....

It is a fact as stated by the committee that there is no method known by which we can get a quantitative estimation of all the living bacteria in a sample. Doubtless a large number of the bacteria which grow at 37.5° C. are closely related to the pathogenic types. That this number is large enough in proportion to cause a definite fluctuation in count directly comparable to the sanitary quality of the water was a question about which the writer was very skeptical. It was this sincere doubt together with lack of definite information on the interpretation of results from agar at 37.5° C. that made him hesitate to discard, point blank, the gelatine method with which he had had years of experience for the agar method with which he had had only unsatisfactory experience and which had apparently so little data behind it.

We knew not what it really meant in terms of gelatine or coli for general analytical work. We were further ignorant of its real value in the estimation of purification efficiency. If you should get 100 per cent removal, what does it mean? It means undoubtedly a zero count on the purified water and that appears good. On the other hand, suppose you have a calculated reduction of 80 per cent; does that mean the purified water was high or the raw water low? It does not necessarily mean low efficiency as efficiency should be reckoned.

All who are familiar with purification works know well that a statement of the percentage removal is about as useless a thing as anyone ever took time to calculate. Actual and comparable counts of the before and after treatment are the only figures upon which we can place any dependence. Even these are very insufficient when it comes to passing final judgment on the value of the water from a sanitary standpoint.

The Colon group of organisms is, at present, the best indication we have of the sanitary condition of the water. Since this is true, the writer took the results of the Colon test as the basic measure in studying the relative value of agar and gelatine counts. All data is presented with this idea in view. It should be stated, however, that the factors affecting bacterial flora in any surface water are many. Temperature, seasonal, climatic, topography, geology and arability of the water shed are among the usually most important. In the case of the western Pennsylvania water, we may add industrial and trades wastes and the acid streams of the mine regions and the result brought about by a conglomerate mixing of these with an uncertain

alkalinity from limestone sections. This brings us to a point where we are ready to discuss the data presented.

A study of Tables 1 to 4, inclusive, brings out no definite relation between the gelatine and coli, the agar and coli or the gelatine and agar. Of course there are certain apparent relations, but they show more exceptions than rules. They vary seasonally and they vary with the classes of water. It is not in the least surprising that this should be so.

In Table 5 the true averages for the entire sixteen months, for the various classes of water, and coli, conditions are given. If any useful relation exists it should be here. In all of these tabulations, the writer believes that the choice is for the gelatine rather than the agar.

Tables 6 and 7 go a step further and give the ratio of agar to gelatine for each class of water and coli condition. In addition the average monthly temperature of the water is given. It is here that there is a marked relation.

In general, the high ratios go with the high temperatures and the low ratios with the low temperatures. This may be partly explained by the fact that in the other tables the agar and gelatine counts fluctuate in opposite direction in respect to temperature.

That the gelatine counts go up as high as shown in cold weather is due in no small way to the existence of low temperature organisms of soil origin. On the other hand, the agar count naturally falls off in the cold weather and goes up in warm weather. It does not follow however that the agar count is higher at the period of highest concentration of pollution, in fact, the data show that it is not.

It is a well known and accepted fact that there are many 37° bacteria in soil that have no sanitary significance. The data here presented bear this out.

Low agar counts on effluents at filter plants in cold weather, would indicate improved working conditions, yet every one knows that such is never the case.

The data presented is perfectly frank in withholding any definite relationship but does show a good seasonal variation. The writer sees no necessity for his further discussion of this data at this time. However, there are certain other points concerning the question of replacement of the gelatine by the agar count which should be taken up.

TABLE 5

Comparison of gelatine and agar counts for various Coli conditions. Counts shown here are the average for sixteen months

B. COLI	CLASS A WATER			CLASS B WATER			CLASS C WATER			CLASS D WATER		
	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar	No. of samples	Gelatine	Agar
Negative in 10 cc							113	920	30	352	44	8
Positive in 10 cc							194	1,075	43	75	571	29
Negative in 1 cc	14	4,741	367	36	9,313	845						
Positive in 1 cc	52	46,590	3,386	113	42,982	5,420	71	1,655	56	11	715	16
Positive in $\frac{1}{10}$ cc	102	45,511	11,728	166	35,641	6,443	13	3,297	26			
Positive in $\frac{1}{100}$ cc	94	37,762	11,177	143	33,106	6,345						
Positive in $\frac{1}{1000}$ cc	93	35,677	9,129	101	31,727	6,821						
Positive in $\frac{1}{10000}$ cc	104	49,175	19,393	56	38,632	11,680						
Total samples	459			615			391			438		

TABLE 6

Showing ratio of agar to gelatine counts for Class A and Class B waters together with their average monthly temperature, Centigrade

	TEMP. CENT.	NEGATIVE 1 cc.		POSITIVE 1 cc.		POSITIVE $\frac{1}{10}$ cc.		POSITIVE $\frac{1}{25}$ cc.		POSITIVE $\frac{1}{50}$ cc.		POSITIVE $\frac{1}{100}$ cc.	
		Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B	Class A	Class B
1912													
October	14	.....	.....	0.189	0.778	0.299	0.212	0.177	0.181	0.364	0.160	0.264	0.068
November	7	.....	0.012	0.333	0.040	0.234	0.048	0.124	0.030	0.123	0.078	0.103	0.025
December	2	.....	.....	0.042	0.018	0.725	0.046	0.041	0.020	0.043	0.012	0.014	0.032
1913													
January	3	.....	0.032	0.055	0.022	0.036	0.034	0.051	0.031	0.056	0.027	0.064	0.044
February	1	.....	0.008	0.056	0.007	0.020	0.006	0.018	0.016	0.044	0.026	0.018	.....
March	4	.....	0.012	0.022	0.011	0.032	0.031	0.042	0.048	0.039	0.019	0.046	.....
April	8	.....	.....	0.134	0.087	0.317	0.055	0.065	0.117	0.093	0.049	0.075	0.023
May	16	.....	.....	0.264	0.338	0.258	0.194	0.293	0.168	0.089	0.192	0.163	0.274
June	21	.....	1.83	.....	0.508	0.399	0.480	0.532	0.218	0.431	0.443	0.627	.....
July	24	.....	.....	.....	.....	0.400	0.833	0.602	0.677	0.423	0.592	0.611	0.571
August	25	.....	.....	.....	0.890	0.773	1.31	0.659	1.30	0.509	0.747	0.816	1.12
September	20	.....	0.396	0.198	0.201	0.699	0.788	1.02	0.843	0.622	0.734	0.772	1.08
October	15	.....	0.136	0.213	0.196	0.675	0.236	0.338	0.259	0.249	0.538	0.467	0.530
November	8	.....	.....	0.168	0.024	0.042	0.070	0.042	0.037	0.036	0.028	0.020	0.026
December	4	.....	0.020	0.326	0.015	0.021	0.016	0.029	0.043	0.008	0.006	0.016	.....
1914													
January	1	0.023	0.016	0.887	0.013	0.078	0.027	0.086	0.052	0.019	0.056	.....	0.199

TABLE 7

Showing ratio of agar to gelatine counts for Class C and Class D waters, with their average monthly temperature, Centigrade

	TEMP. CENT.	NEGATIVE 10 cc.		POSITIVE 10 cc.		POSIT.VE 1 cc.		POSITIVE $\frac{1}{10}$ cc.	
		Class C	Class D	Class C	Class D	Class C	Class D	Class C	Class D
1912									
October	15	0.197	0.721	0.243	0.148	0.178	0.219	.....	.....
November	8	0.040	0.131	0.052	0.013	0.060	0.131	0.001	.....
December	3	0.009	0.183	0.034	0.017	0.020	0.023	0.018	.....
1913									
January	4	0.135	0.320	0.062	0.060	0.037	.....	0.028	.....
February	2	0.006	0.196	0.007	0.046	0.004	0.083	.....	.....
March	5	0.253	0.364	0.088	0.318	0.029	.....	0.023	.....
April	9	0.171	0.727	0.092	0.355	0.600	.....	0.012	.....
May	17	0.051	0.500	0.174	0.268	.....	.....	.....	.....
June	21	0.079	0.500	0.138	0.089	.....	.....	.....	.....
July	25	0.257	0.642	0.370	0.379	0.209	.....	.....	.....
August	25	0.644	0.750	0.810	5.43	1.12	.....	.....	.....
September	21	1.44	0.769	0.672	0.837	0.835	.....	.....	.....
October	16	0.246	1.33	0.256	1.70	0.209	0.045	0.065	.....
November	8	0.030	0.096	0.030	0.010	0.009	0.005	0.007	.....
December	4	0.003	0.012	0.011	0.042	0.003	.....	.....	.....
1914									
January	2	0.002	0.014	0.009	0.029	0.001	.....	0.016	.....

There is practically no difference in the ease or difficulty with which the two media are made up. The one has no advantage over the other. The agar at 37° C. has an apparent advantage in that it will give a count at the end of twenty-four hours. On the other hand, it has the disadvantage of giving many lost results due to troublesome spreaders even with the porous cover dishes. The writer finds that the loss by liquefying of the gelatine plates is practically nil, hence the results on gelatine, while they take longer, are more certain.

It has been the writer's experience that the gelatine count will frequently indicate conditions which would never be shown by the agar 37° count alone. Such conditions as an abnormal fluctuation in a reservoir, breaks or other abnormal drafts on pipe lines, abnormal growths in filter underdrains, springs and wells sensitive to local rains through shallow drainage and other similar problems which many have doubtless encountered. These may not have an immediate direct bearing on the public health, yet they are conditions which we wish to detect.

We are not getting full efficiency from our bacteriological tests unless we do. The agar 37° count in connection with the gelatine often emphasizes these conditions but many of them it would never detect alone. Furthermore, the agar 37° count on the better class of waters is so low that it would take considerable audacity to say that there was a different interpretation to be placed on a fluctuation of 5 or 10 per cubic centimeter.

One hundred per cubic centimeter or less on gelatine at 20° is the often quoted German standard for good water This is no doubt reasonably correct, yet we have seen waters with this low count which were still suspicious. On the other hand, we have seen waters with higher counts which we felt were reasonably good.

The agar 37° count which corresponds to this is probably somewhere between 0 and 100, but where? The writer has not the courage to fix the standard from any data in his possession. In fact the more he studies bacteriological data the more convinced is he that the setting of fixed standards is poor policy. Results should be interpreted from knowledge of all related conditions by a man of experience in these matters.

Take Tables 3 and 4, the gelatine shows a markedly wider range of fluctuation and consequently is a much better indicator of working conditions. On the other hand, the agar has no advantage as an indicator of sanitary conditions.

In conclusion, the writer wishes to add his word of protest against the replacement of gelatine 20° C. counts with agar 37° C. counts as standard procedure in water analysis. If it is possible to make only one count the gelatine should be used as it is more reliable and servicable.

If possible, both gelatine counts at 20° C. and agar at 37° C. should be made as it gives to the experienced man more data for his diagnosis and points will often be brought out more completely when we have the two counts.

The value of the two counts over the one will be constantly increasing as we become better acquainted with interpretation of such results.

SEWAGE POLLUTION OF BOUNDARY WATERS

By Allan J. McLaughlin, M.D.[1]

In December, 1910, the writer was directed by the Surgeon-General of the Public Health Service to investigate the sewage pollution of interstate and international waters, with special reference to the spread of typhoid fever.

In 1911, he completed a sanitary survey of the entire watershed of the Great Lakes on the United States side of the boundary.[2]

Briefly the conditions found were as follows:

An excessive prevalence of typhoid fever especially in the winter and spring months, punctuated at intervals by explosive epidemics. This excessive prevalence of typhoid fever especially in the winter and spring months was due in greatest measure to the unrestricted discharge of sewage into interstate and international waters used as sources of public water supplies. Disaster followed the use of this sewage polluted water for one of two reasons, either a failure to purify or the inefficiency of the attempted purification.

The delusion that water from the Great Lakes or their connecting rivers needs no purification has been cherished for years in our cities and even with our disgraceful record of waterborne typhoid and the lessons of numerous disastrous epidemics, it is still no easy task to convince municipal officials that purification of these waters is necessary.

The remedies suggested by the writer at that time were:

1. Safe water supplies, that is, water shown to be safe by daily bacteriologic examination.

2. Supervision and control of water supplies by the state to ensure efficiency and a safe effluent at all times.

3. Control of sewage discharge within permissible limits to prevent an unreasonable burden or responsibility upon filter plants.

[1] Surgeon, U. S. Public Health Service, Chief Sanitary Expert and Director of Field Work for the International Joint Commission.

[2] McLaughlin, Allan J., Sewage Pollution of Interstate and International Waters. Hygiene Laboratory Bulletins 77, 83, and 89, Treasury Department, Washington, D. C.

4. Prevention of pollution from vessels.

In order to secure efficient and uniform results from the application of these remedies, he recommended that two sets of standards for water be formulated.

1. Standards for filtered or treated water.
2. Standards for raw water at the intakes.

In accordance with this recommendation in January, 1913, the Surgeon-General appointed a Commission for the Determination of a Standard of Purity for Drinking Water, and the report of that Commission will soon be published. This report will furnish us with a minimum standard to which all common carriers, trains and vessels must conform to prevent the spread of disease in interstate traffic. This furnishes the first standard recommended. The fixing of the second standard or standard for raw water at the water works intakes is a much more complex problem. However, great strides have been made in this direction also.

A committee of the National Association for the Prevention of Pollution of Rivers and Waterways,[3] made up of Geo. C. Whipple, Edward Bartow, Geo. M. Wisner, H. W. Clark, and the writer studied the problem of standards in a general way and agreed upon certain fundamental principles which made a very good starting point for more intensive studies. This brings us to the work of the International Joint Commission.

The term Commission is misleading as this body is in reality a tribunal formed under the treaty of January 11, 1909, between Great Britain and the United States to prevent disputes regarding the use of the boundary waters and to settle questions now pending or which may arise hereafter between the United States and Canada, involving the rights, obligations or interests of either in relation to the other or the inhabitants of the other along their common frontier. In addition to their judicial powers the Commission under Article IX of the treaty is empowered to investigate and report upon such questions as may be referred to it by the two governments. It is provided that such reports under Article IX shall not be regarded as decisions of the questions and shall in no way have the character of an arbitral award under this investigative function.

The question of pollution of boundary waters was referred to the Commission by the two governments. The reference was in two

[3] Pollution of Rivers and Waterways, Senate Document 235, 63d Congress, Washington, D. C., 1913.

sections, the first dealing with the location, origin and extent of pollution and the second dealing with remedies. The text of the reference was as follows:

> 1. To what extent and by what causes and in what localities have the boundary waters between the United States and Canada been polluted so as to be injurious to the public health and unfit for domestic or other uses?
>
> 2. In what way or manner, whether by the construction and operation of suitable drainage canals or plants at convenient points or otherwise, is it possible and advisable to remedy or prevent the pollution of these waters, and by what means or arrangement can the proper construction or operation of remedial or preventive works, or a system or method of rendering these waters sanitary and suitable for domestic and other uses, be best secured and maintained in order to insure the adequate protection and development of all interests involved on both sides of the boundary, and to fulfill the obligations undertaken in Article IV of the waterways treaty of January 11, 1909, between the United States and Great Britain, in which it is agreed that the waters therein defined as boundary waters and waters flowing across the boundary shall not be polluted on either side to the injury of health or property on the other?

The Commission took up first the question of location, origin and extent of pollution. At the Detroit session February 20, 1913, the writer was placed in charge of the investigation as chief sanitary expert and director of field work. Associated with him on the Canadian side were Dr. John W. S. McCullough, Chief Health Officer of Ontario and Dr. John A. Amyot, Professor of Hygiene, University of Toronto. Canadian laboratories were established at Fort Frances, Port Arthur, Sault Ste. Marie, Sarnia, Windsor, Amherstburg, Port Stanley, Fort Erie, Niagara-on-the-Lake and Kingston, Ontario, and Montreal. American laboratories were established at Port Huron, Michigan, Detroit, Michigan; on U. S. Revenue Cutter *Morrill*; Buffalo, New York; Youngstown, New York; Clayton, New York; and Van Buren, Maine.

In the interest of economy and efficiency it was deemed wise to utilize such established governmental agencies as might be available for the field investigation. Accordingly the coöperation of the United States Public Health Service, the Provincial Boards of Health of Ontario and Quebec, the State Board of Health of Michigan, and the Department of Health of the State of New York was secured and was of very great value to the Commission in carrying out the work.

The work extended from the Lake of the Woods through the entire chain of the Great Lakes and St. Lawrence River to the St. Johns

River on the Maine—New Brunswick border. Field work began in April and terminated November 1. There were over 1400 sampling points and over 19,000 samples taken and examined bacteriologically. The report presents as indices of pollution the average number of the colon bacillus per 100 cc. of water and the number of bacteria per cubic centimeter of water. The results of the investigation are shown by averages in concise form for each sampling point and cross-section by tables and colored maps. In addition diagrammatic representation is employed to show the degree of intensity of the area of pollution in each section of the boundary waters. In certain localities on the Great Lakes and in all their connecting waters dangerous sewage pollution was shown to exist, but the bulk of the Great Lakes waters remains in its pristine purity. The investigation shows that the colon bacillus is practically never present in unpolluted waters, and that the normal bacterial content of Great Lakes water is less than 10 per cubic centimeter. Great Lakes water is classified tentatively into five classes in this report, from the relatively pure water through slight, moderate, serious to gross pollution. The sources of pollution in the order of their importance are sewage from cities, sewage from vessels navigating these boundary waters, and the inevitable pollution following rains and thaws. The distance pollution may travel in the Lakes was demonstrated also. At the mouth of the Detroit River and at the mouth of the Niagara River serious pollution extends normally more than 10 miles into the lakes and on occasion was found 16 to 18 miles from shore. The distances from the cities of pure water in the lakes, the enormous cost of long pipe lines, coupled with the engineering difficulties in placing intakes beyond a 70-foot depth, make it impracticable in most instances to secure pure water from the lakes without treatment. The present position of intakes is such that there is not a single municipality using lake water which can be said to possess a safe water supply without treatment.

As might be expected the areas most grossly polluted are in the connecting rivers upon which large cities are situated. The pollution from vessels renders the St. Mary's River above the cities of Sault Ste. Marie unfit as a source of water supply without treatment, and vessel pollution combined with the sewage from the cities of Sault Ste. Marie cause gross pollution of the St. Mary's River from the Sault to Neebish Island.

Because of sewage pollution there is no point in the St. Clair River from which a safe water supply could be secured without treatment.

The Detroit River is polluted from Lake St. Clair to Sandwich sufficiently to make the water an unsafe source of water supply. From below Sandwich to its mouth the Detroit River is grossly polluted from shore to shore.

Gross pollution in the Niagara River extends along the American shore from Buffalo to Strawberry Island, throughout the entire Tonawanda Channel, and below the Falls gross pollution extends from shore to shore throughout the entire river and for miles out in Lake Ontario.

Investigation of the St. Lawrence River (Lake of the Thousand Islands) was made at two distinct seasons. The results obtained in April showed a slight degree of pollution. The results obtained in August showed a greatly increased pollution obviously due to the increase in summer resort population and to the sewage discharged from vessels.

The report of the field work gives an accurate survey of the origin, location and extent of pollution and completes the first half of the reference. The Commission is about to take up the second half of the reference which relates to remedies for pollution.

The writer's suggestion to the Commission in regard to proceeding with the second part of the reference consisted of three steps.

1. Secure the opinions of the most eminent sanitary engineers by submitting a list of questions. These opinions would form a sound basis for formulating a general policy and possibly some general minimum requirement.

2. Public hearings at which the municipalities could present their cases including expert engineering testimony, plans and projects.

3. Employment of expert sanitary engineers for the formulation of special standards for each waterway in excess of general minimum requirements depending upon local conditions and necessities.

It is probable that the Commission will proceed somewhat according to this outline. In so doing and in carrying out the third step, formulation of special standards, we will realize the second of the essentials which are necessary for the protection of the public health, viz: A standard for the raw water at the intakes.

The standard for raw water at the intakes in each locality seems to be the only means by which an equitable adjustment can be secured between the two agencies in producing safe water, viz: water purification and sewage treatment.

The writer is very glad to have the honor and opportunity of presenting this paper on sewage pollution of boundary waters to the American Water Works Association because of the special interest which this subject has for cities in the basin of the Great Lakes and the general interest which it must excite because of kindred problems in other watersheds. The writer cannot resist the temptation to avail himself of this opportunity to accentuate the necessity for safe water supplies. Time will not permit him to dwell at length upon certain cardinal necessities in the purveying of water to the public but he asks your permission to at least enumerate these points for your consideration.

The writer is glad to say that water works men generally appreciate that their duty obliges them to furnish not only sufficient water and clear water but most important of all safe water. It also gives him much pleasure to state that as a rule the water works engineers and superintendents of today are quite as much interested in the securing of a safe effluent as in utilizing the greatest amount of heat from their fuel or any other of their mechanical problems. Now what is a safe water? A safe water is one which is by daily bacteriologic examination shown to be free from pathogenic or intestinal organisms.

There has been entirely too much confidence in untreated surface supplies. Remember that a surface supply from an inhabited watershed is rarely safe without treatment and can only be said to be safe when daily bacteriologic examination shows it to be free from pathogenic or intestinal germs. To ensure safe water bacteriologic control is just as much a necessity for water purification plants as for untreated supplies. With a polluted source the mere installation of a purification plant does not guarantee safe water. Even if perfect in design and construction unless efficiently operated and controlled a safe effluent need not be expected. The writer has seen filter plants designed by our best engineers—perfect mechanisms which if properly operated would produce safe water—placed in the hands of an assassin who was a promoted stoker and absolutely ignorant of bacteriology or chemistry.

There is another point the writer desires to make and this refers to the adjustment of the balance between water purification and sewage treatment. He has seen very good filter plants struggling with a raw water which imposed an unreasonable responsibility upon the plant. A safe effluent under such conditions was secured at the price

of eternal vigilance. Now, filter plants are not infallible and their operators even when skilled are only human. The writer believes a decent raw water should be made available for all plants and where necessary treatment of sewage should be carried at least far enough to secure this result.

The greatest obstacle to proper operation and control of plants has been the difficulty of securing the right man to place in charge of the plant. The best type of man for this position is a graduate in sanitary engineering. He will not only be conversant with the mechanical details of the plant, but will be able to adjust his chemicals according to the constituents and needs of the raw water. Most important of all, he will be able to make daily bacteriological examinations to determine the efficiency of purification. Nearly all the disasters due to sewage polluted water supplies which have occurred were due to lack of daily bacteriological knowledge of the public supply or the inefficient operation of plants by unskilled men. The writer believes the employment of such a graduate is economy even in small cities. He can conceive, however, of cases where it is impossible for economic reasons to pay the necessary salary. In these cases local men must be employed, and trained to do the work. Here the State Board of Health, or as in Illinois the State Water Survey, will find a very useful function. The State authorities could supervise the installation of a small, inexpensive laboratory equipment in small plants and give instruction to the local man in making the necessary water examinations. Whenever possible, however, young graduates of sanitary engineering schools should be employed; and such men are well worth their salary considering the saving in the economical adjustment of chemicals and fuel costs made possible by intelligent supervision. The greatest asset, however, to be credited to skilled operation is the saving of human life effected and the satisfaction of knowing that safe water is being furnished every day.

PRESENT DAY WATER FILTRATION PRACTICE

By George A. Johnson

It is now eighty-five years since the first municipal water filter was built at London, and this filter was built to perform only the functions of a mechanical strainer for the removal of suspended matter. The two most important water-borne diseases, typhoid fever and cholera, had not then been discovered, nor, indeed, was the germ theory of disease seriously advanced until some twenty years later. Doubtless even the earliest water filters were of considerable sanitary benefit, but the first official recognition of water filtration as a means of reducing the dangers in impure drinking water followed the severe cholera epidemic of 1849. This took the form of an act of Parliament (1852) which made compulsory the filtration of the entire water supply of the Metropolitan District of London.

Typhoid fever as a specific disease was discovered about 1829, and Koch discovered the cholera spirillum in Calcutta water in 1883. In 1889 Brouardel attributed 90 per cent of all cases of typhoid fever to contaminated water, but it was not until 10,000 lives had been sacrificed during the Hamburg cholera epidemic of 1892 that the conviction spread broadcast that impure water was surely responsible for this stupendous epidemic, the happy experience of Altona, the sister city of Hamburg, furnishing unmistakable proof of the efficacy of water filtration.

The first filter at London, as well as all those that followed during the succeeding fifty years, was of the slow sand, or English type. This type of filtration was the subject of much early scientific investigation, particularly in Germany, and was brought to this country in the late sixties by James P. Kirkwood who, in the middle seventies, designed and built at Poughkeepsie, New York, the first municipal water filtration plant in America. Other small plants of the slow sand type were built in this country shortly afterwards, but it is somewhat remarkable that as late as 1880 only 30,000 people in America were being served with water so filtered, and in 1890 but 5000 more.

In 1884 a patent was granted to Isaiah S. Hyatt for a process in which a coagulant was added to the raw water before filtration. This

opened a new era in the art of water purification, although the idea of coagulating water to accelerate clarification was practiced by the Chinese thousands of years ago. Indeed there is a reference in the Holy Writ which seems to refer to this same idea (II KINGS, 2, 19–22).

With the use of coagulating chemicals there developed an entirely new type of water filter, which became known as the American, mechanical, or rapid sand filter, as distinguished from the English, or slow sand filter. Today, although there are variations from the parent types, the two recognized methods of water filtration are slow sand filtration and rapid sand filtration.

It is not necessary to describe these two methods of water filtration. The object of this paper is rather to trace the development of each type; to compare their relative applicability to meet varying sets of conditions; and, since neither can be operated automatically, to discuss certain features of operation upon which the efficiency of both types depend; and, also, to dwell somewhat on questions of the comparative cost of filtered water obtained by the two methods.

GROWTH OF SLOW SAND FILTRATION IN THE UNITED STATES

Between 1875, when the first American slow sand filter was built at Poughkeepsie, New York, and January 1914, some thirty municipal filters of this type were put in operation or are now under construction; and at this date such plants have a daily filtering capacity of about 840,000,000 gallons, and are designed to serve a total population of some 5,500,000. Of this population 721,000, or about 12 per cent of the total is located in New England, where Providence, Rhode Island, New Haven, Connecticut, Lawrence and Springfield, Massachusetts are the largest cities having slow sand filter plants. Of the remaining population in the United States served by slow sand filter plants the bulk is centered in the cities of Albany, New York, Philadelphia, Pittsburgh and Reading, Pennsylvania, Washington, District of Columbia, Wilmington, Delaware, Indianapolis, Indiana, and Denver, Colorado. The present population of these eight last named cities is about 4,000,000, representing about 73 per cent of the total. The remainder of the population served by this type of filters, equivalent to about 15 per cent of the total, is widely scattered through some twenty cities.

Lawrence, Massachusetts. In a relatively large number of instances local preliminary experiments were conducted ahead of the actual

construction of filter plants. Such studies were carried on at Lawrence for some five years prior to the construction of the Lawrence City filter, which was built in 1893 and enlarged in 1907. Slow sand filtration was the only method tested, as rapid sand filtration had but little standing at that time. This plant has done excellent work in the purification of the polluted, but quite clear Merrimack River water, the filters being operated at rates of filtration of about 3,000,000 gallons per acre daily. The old filter is uncovered, and has always given trouble during cold weather on account of the accumulation of ice on the surface, making cleaning operations difficult, and at times impossible. Neither of the filters is equipped with rate controlling apparatus, and it is therefore impossible accurately to tell at a given time at just what rate the filter is actually operating.

The reduction in the typhoid fever death rate in Lawrence, following the installation of the city filter, was very marked. Whereas for the twenty years preceeding 1894 the average annual death rate in Lawrence from this disease was 109 per 100,000 population, the annual average for the last twenty years with filtration was 23 per 100,000, a reduction of 79 per cent.

Albany, New York. This slow sand filter plant, constructed in 1898–99, was, at the time it was built, the largest vaulted masonry filter plant in the United States. It was designed to furnish 15,000,000 gallons of filtered water daily. The water was allowed some two days plain sedimentation before being applied to the filters, but as the Hudson River is a semi-muddy stream, at times becoming very muddy for short periods, such preparatory treatment was found to be inadequate and a battery of roughing filters was installed in order to bring about a more complete removal of suspended matter before the water was applied to the final filters. The necessity for this step is apparent from the following figures which show the average range in turbidity of the raw Hudson River water at Albany. The average annual turbidity is about 40 parts per million.

Range in turbidity of Hudson River water at Albany, New York

RANGE IN TURBIDITY (PARTS PER MILLION)	NUMBER OF DAYS	PER CENT OF ENTIRE YEAR
50	55	15
100	25	7
200	10	3

Formerly, before the preliminary roughing filters were added to the original layout, the filtered water was sometimes noticeably turbid, and the bacterial efficiency, particularly during cold weather, fell off markedly. Of late years the sterilization of the final product with hypochlorites has resulted in a filtered water of first class bacterial quality; but since the river water contains some 30 parts of color on an average, and filters of this type are unable completely to remove such color, the filtered product frequently is more or less unsatisfactory in this respect, and will continue to be so unless coagulating chemicals are used for the removal of such vegetable stains.

The efficiency of the Albany filters in reducing the typhoid fever death rate in that city has been very marked. For the ten years previous to the construction of the filtration plant the average death rate from this disease was 90 per 100,000 population; and for the 13 years ending 1912 the average rate was 21, a reduction of about 77 per cent.

Washington, District of Columbia. This 100,000,000 gallon slow sand filter plant, completed in 1906, is one of the largest of its type in the United States. Before the existing works were placed under construction careful preliminary studies were made to determine the relative applicability of slow sand and rapid sand filters to the purification of the Potomac River water. Lieutenant Colonel A. M. Miller, in reporting the results of these studies to Brigadier General John M. Wilson, Chief of Engineers, U.S.A., recommended that for the filtration of the Washington water supply the rapid sand filtration system should be adopted, for the following reasons:

> These investigations have continued during a period of nine months. During this period the Potomac water has passed through extreme stages, from clear to very turbid. With the exception of a few days it has been very turbid from December 28, 1899, to March 13, 1900.
>
> During this period the English (slow sand) filter has not furnished a satisfactory effluent, either as to turbidity or bacterial efficiency.
>
> During the same period the American or mechanical (rapid sand) filter has furnished an effluent which was entirely satisfactory, both as to turbidity and numbers of bacteria present.
>
> At no time has there been any indication of sulphate of alumina present in the effluent of the mechanical (rapid sand) filter.
>
> It is therefore demonstrated that during the period of greatest turbidity and accompanying bacterial danger the English filter will not furnish a satisfactory effluent; while, on the contrary, during this period the mechanical (rapid

sand) filter will, with proper attention, furnish an entirely satisfactory effluent both as to turbidity and sanitary considerations.

Colonel Miller's recommendations were not carried out, one of the reasons being the existence in Washington of a more or less popular prejudice against the use of a coagulating chemical in connection with the purification process. Early in 1901 an expert commission recommended the adoption of the slow sand filter process

> . . . . With such auxiliary works as may be necessary for preliminary sedimentation, and the use of a coagulant for a part of the time. There is no reason to believe that the use of this coagulant will in any degree effect the wholesomeness of the water.

When the bill providing for the construction of the filtration plant was finally passed it did not include an appropriation for the coagulant but which in recent years has been used.

The Potomac River is somewhat peculiar, in that the turbidity of its waters ranges from some 3000 to practically nothing; but it is the practice to close the inlet gates on the river at Great Falls when the turbidity of the river water reaches 500 or more. In the year 1912 the gates were closed some 21 per cent of the time, and by this means some 30 per cent of the total suspended matter, that would otherwise have entered the purification system, was excluded.

The reservoir system, through which the water passes on its way to the filters, provides a total period of sedimentation of some six or seven days. Since 1911 sulphate of alumina has been applied to the water in the middle reservoir (Georgetown) on about one-third of the days in the year. On some occasions the filtered water possesses a noticeable turbidity, but on account of the present complete preparatory treatment it is normally free from turbidity and the bacterial efficiency high. The efficiency of the purification plant is well shown by the figures given in the following table:

Efficiency of Washington water purification system

Turbidity

	RESERVOIR OUTLET	1907	1908	1909	1910	1911	1912	AVERAGE FOR SIX YEARS
Maximum	Dalecarlia	250	340	200	200	120	700	300
	Georgetown	150	250	170	215	70	70	155
	McMillan Park	250	160	85	120	40	40	116
	Filtered water	13	20	8	9	1	4	9
Average	Dalecarlia	46	53	50	30	18	59	43
	Georgetown	37	45	32	29	16	23	30
	McMillan Park	29	31	22	18	10	13	22
	Filtered water	2	2	1	1	0	0	1

Bacteria

Maximum	Dalecarlia	24,000	65,000	12,000	175,000	49,000	119,000	74,000
	Georgetown	23,000	52,000	13,000	180,000	25,000	20,000	52,000
	McMillan Park	15,000	19,000	3,500	180,000	25,000	10,100	44,000
	Filtered water	200	900	160	2,800	300	460	800
Average	Dalecarlia	1,940	2,700	1,950	13,850	3,370	6,000	4,970
	Georgetwon	1,680	2,940	950	10,850	2,080	2,600	3,350
	McMillan Park	635	1,250	390	6,820	1,390	1,100	1,930
	Filtered water	31	55	21	143	38	35	54

The typhoid fever reduction in Washington has not been so marked as that noted in other cities following the filtration of the water supply. For the six years prior to the construction of the filtration plant at Washington the average death rate from this disease was 57 per 100,000 population; and for the following six years ending 1912 it was 31, a reduction of 54 per cent. In view of the good work done by the filter plant it is probable that no inconsiderable part of the typhoid fever in Washington during this latter period came from other sources than impure water. It is a fact that there has been a more or less steady annual decrease in that disease in Washington since the filter plant was put in operation, the death rate by years for the period 1907–12 being 36, 39, 33, 25, 22, 23, respectively, per 100,000 population.

Philadelphia, Pennsylvania.—A long series of preliminary studies were carried on in Philadelphia before and during the early stages of construction of the filtration system of that city. Although the Schuylkill and Delaware Rivers, from which the water supply is taken, are to be classed among the medium muddy waters, the experimental studies were confined to slow sand filtration systems, it being decided well in advance of the commencement of actual design of the works that the rapid sand process should be eliminated from consideration.

About 65 per cent of the present supply is taken from the Delaware River and purified at the Torresdale plant which has a daily capacity of 250,000,000 gallons. The remaining four plants, at Upper Roxborough, Lower Roxborough, Belmont and Queen Lane, respectively take water from the Schuylkill River, and have a combined daily capacity of 142,000,000 gallons. At the three last named filter plants the water is prepared for slow sand filtration by plain sedimentation and filtration through preliminary filters in order to remove as much as possible of the suspended matter in the raw water. Plain sedimentation only is used at the Upper Roxborough Plant.

That the introduction of filtration plants at Philadelphia has been the direct cause of a marked reduction in typhoid fever in that city there can be no doubt for whereas for the eight years ending 1907 the average typhoid fever death rate was 54 per 100,000 population, for the five years that followed (1909–12) the average was 22, the averages for the individual years being 36, 22, 17, 14 and 12, respectively. The materially lower typhoid fever death rate in recent years may be, and probably is, in some measure due to the use of hypo-

chlorites to sterilize the final filtered product, as is also the practice at the Albany slow sand filter plant. The first filter contract was let in 1901, and the various plants were finished and placed in operation one after another during the ten years that followed, a greater proportion of the population being supplied as each successive plant went into service. This also accounts for the consistently decreasing typhoid fever death rate.

In connection with the applicability of slow sand filters to the treatment of such clay bearing waters as the Delaware River, some very instructive information was furnished in a paper read before the last annual convention of the New England Water Works Association, held in Philadelphia in September, 1913. In this paper, and in an entirely impartial and unusually candid fashion, there were set forth, the actual difficulties under which a slow sand filter is forced to labor when called upon to purify a water the like of which no slow sand filter should, in the light of past experience, be expected continuously to treat in an efficient and economical manner.

The Torresdale filter plant when built was supposed to be the *dernier cri* in slow sand filter design and construction. No preliminary sedimentation basins were provided at this plant, but 120 preliminary filters, in general of the rapid sand filter type of construction, and having a total filtering area of $1\frac{1}{3}$ acre, were expected adequately to prepare the raw Delaware River water for filtration through the final slow sand filters. The filtering material in these preliminary filters consisted of 15, 3 and 8 inches, respectively of graded gravel, the bottom layer varying in size of particles from 2 to 3 inches and the top layer from $1\frac{1}{8}$ to $\frac{1}{4}$-inch; and on top of the gravel layer was placed a 12-inch layer of sand varying in size of particles from 0.8 to 1 mm. In all important respects the Torresdale preliminary filters, in design and construction, are rapid sand filters. And they are operated like rapid sand filters except in the very important respect to coagulation of the water prior to its application to them. It was the original idea that no coagulants were to be used at any of the Philadelphia filter plants, but their use there is now under serious consideration, if, indeed they are not already being used.

Experience has shown that slow sand filters begin to work disadvantageously when the water applied to them contains for several successive days more than 30 to 50 parts of fine turbidity. At such times the effluent often deteriorates markedly in quality, not only from the standpoint of appearance but also bacterially.

In the year 1912 the water delivered by the Torresdale preliminary

filters to the final slow sand filters had a maximum turbidity of 610 parts per million, and on twenty days the turbidity was in excess of 50 parts per million. Incidentally the effluent of the slow sand filters possessed a turbidity of over 10 parts on nine days, and a turbidity of this intensity is clearly unsatisfactory.

Plain sedimentation and roughing filters will remove much of the turbidity from a muddy water, but, unhappily for those who do not favor the use of a coagulant, the residual turbidity in the effluent of these preparatory processes is caused by suspended matter in a state of fine subdivision. This matter, when applied to slow sand filters, penetrates deeply, indeed in some cases passing completely through the filter and out in the filtered water, to the detriment of the appearance and bacterial quality thereof.

In the paper above referred to the authors state that the Torresdale plant as it exists today, although, by virtue of the use of hypochlorites producing a water of relatively high bacterial purity, can be depended upon to yield a clear effluent but 48 weeks in the year. When the turbidity of the Delaware River water for prolonged periods is in excess of 100 parts per million the existing plant is "utterly unable to cope" with the problem of yielding a satisfactory effluent. At such times the prefilters fail to do their proportion of the work, and the final slow sand filters choke badly and allow fine silt to pass through them. The penetration of suspended matter into the beds is sometimes as great as 10 inches. Under the most unfavorable conditions, in order to maintain the required yield from the plant, it is necessary to continue cleaning operations for 24 hours in the day, under which circumstances about 85 per cent of the filters are doing 100 per cent of the work. High bacterial removal at such times is due solely to the use of hypochlorite, as before stated.

Pittsburgh, Pennsylvania.—Before a decision was reached as to whether slow sand or rapid sand filters should be adopted at Pittsburgh, comparative studies on an experimental scale were made of both systems. The final conclusion was in favor of the adoption of slow sand filtration, and a plant of this type was placed under construction and completed in 1908.

In the light of events which followed the placing in operation of the filtration plant, and which are dwelt upon at some length beyond, some of the conclusions of the Filtration Commission, in recommending slow sand filtration for Pittsburgh are interesting on account of their failure to be confirmed later by actual practice:

Comparative cost

If a conclusion were to be arrived at upon a consideration of initial cost, alone, the decision would doubtless be prompt. While the cost of bare construction differs but slightly as between the two plans, the cost of the larger body of land, required for the sand filtration plant, would be determinative, in favor of mechanical (rapid sand) filtration, were other things equal.

Efficiency

An examination of the relative efficiency of the two methods, in the light of actual experiment, shows that so far as the removal of bacteria from the water is concerned the (slow) sand filter leaves but little to be desired. In addition to bacterial efficiency and somewhat important, is the question of the adaptability of the effluent for steaming purposes. The effluent which yields a minimum scale formation, and shows no corrosive action upon the points in a boiler generally first exposed to attack, is, all other things being equal, to be preferred. We find that the weight of evidence, obtained by experiment is in favor of (slow) sand filtration; so far as efficiency in this respect is concerned.

Simplicity of operation

In operating a plant of such magnitude as will be required to provide a full supply of pure water to this large and growing city it is of the first importance to do it on such lines as not to require, necessarily, the higher grades of technical skill on the part of a large proportion of the operatives. The (slow) sand filter meets this requirement to such an extent as to be fairly considered ideal. When properly constructed, slight neglects and errors of judgment on the part of workmen cannot damage the water. With any filter using coagulant the conditions are different; nice judgment is required to determine from day to day, and at times from hour to hour, the required quantity of that coagulant, and if there be not the proper quantity of lime in the water, initially, that must also be provided. The choice lies between a system, which, when properly constructed, *cannot* furnish impure water, except as the result of wilful neglect, and one which *can* furnish pure water, but which can also, when carelessly or ignorantly handled, easily produce water unfit for domestic or mechanical use. The weight of evidence is found by the Commission to be decidedly in favor of sand filtration, so far as simplicity of operation is concerned.

As an example of the early efficiency of the Pittsburgh filters which were subsequently built in accordance with the recommendations of the Filtration Commission, let it be stated that in 1909 the records show that the filtered water was ordinarily, but not uniformly, satisfactory in appearance. The average number of bacteria in the filtered water for the eleven months ending December 31, 1909 was 3,900 per cubic centimeter. The highest average number for a single month was 34,300 per cubic centimeter in December, 1909. The

highest number of bacteria in the filtered water for any one day was 120,000 per cubic centimeter, recorded on December 15, 1909. In the face of the above results is it any wonder that the use of hypochlorite was inaugurated on January 3, 1910, for the sterilization of the filtered water?

There are very few filter plants where the details of operation are so carefully watched, or where the varying character of the raw water is so well anticipated and adjusted to, as at the Pittsburgh plant. More to this fact than anything else is due the absence of signal failure of slow sand filtration at Pittsburgh; but it has required the "higher grades of technical skill" to enable the filtration works to handle the most difficult water a slow sand filter plant is called upon to purify in this country.

Next to Philadelphia the Pittsburgh slow sand filter plant is the largest of its type in the United States. As designed the 56 acres of filters were to have a daily filtering capacity of about 140,000,000 gallons as a minimum. By virtue of the highest degree of skilled supervision this expectation was realized, but owing to the fact that the pollution of the Allegheny River with trade wastes is rapidly increasing year by year this would not have held true for very many years without the aid of the preliminary filters and improved sedimentation facilities, such as have been installed during the past two years in anticipation of what is to come. At times the capacity of the filters was severely taxed, and to quote from the report of the Pittsburgh Bureau of Water for 1911: "Without some sort of change in the present system it is certainly true that the economical capacity has already been closely approached on several occasions."

The turbidity of the Allegheny River at Pittsburgh fluctuates from almost nothing to very muddy. The main facts surrounding this point are contained in the following table:

Turbidity of the Allegheny River water at Pittsburgh

1911–12 MONTH	TURBIDITY—PARTS PER MILLION				
	Average	Highest day	Days when turbidity was over		
			50	100	200
February	43	134	10	1	0
March	36	61	7	0	0
April	52	175	13	2	0
May	23	62	2	0	0
June	21	69	2	0	0
July	17	70	1	0	0
August	71	758	7	2	2
September	173	1,469	17	12	5
October	129	1,146	15	9	5
November	49	159	9	2	0
December	53	116	11	2	0
January	24	71	1	0	0
Total and Averages	58	....	95	30	12
Per cent of entire year	...	....	26	8	3

The plain sedimentation reservoirs, having an average period of storage of about thirty-six hours, were the only provision for preparatory treatment in the original design. They were never very efficient in removing suspended matter from the raw water, such removal averaging about 25 per cent through a representative year. Taking the figures in the last table as an example, it is seen that with 25 per cent of the suspended matter removed in these reservoirs the water as applied to the filters during the months of August, September and October would have contained 53, 130, 87 parts per million of suspended matter during these three successive months. On quite a number of occasions the water applied to the filters contained for several successive days at a time from 150 to 250 parts per million of suspended matter. During 1909 the effluent of the sedimentation reservoirs had a turbidity in excess of 25 parts per million for more than half the time, and in excess of 50 parts for some 15 or 20 per cent of the time. Such conditions as these in general could only mean that the final filters would frequently be overloaded and inefficient.

In July 1910, about a year and a half after the works were put into operation, the whole filtration problem at Pittsburgh was placed under

investigation. It was recognized in the beginning that the filters would soon be unable to supply the demand for water, and the studies which followed, besides covering improved methods of operation in interests of economy, were also particularly directed toward a solution of how best to prepare the Allegheny River water for final filtration through the slow sand filters in order to obtain from them a greater net yield of filtered water, and higher bacterial and physical efficiency.

One object of these investigations was to develop methods whereby the acid conditions existing in the Allegheny River water for much of the time could be so modified as to allow of the adequate preparation of the water for slow sand filtration, so that, with improved methods of operation, it would be possible materially to increase the minimum daily yield of the plant, and at the same time obtain a filtered water satisfactory from the standpoints of appearance and bacterial content.

It was learned during the investigations of 1910–11 that speedy clogging of the filters was due primarily to acid conditions in the Allegheny, coming from mine wastes discharged into the river above the intake of the filtration works. The presence in the river water of unprecipitated colloidal hydrates and silicates of iron, aluminum and manganese, complicated at times by the presence of organic coloring matter, parafine like bodies, sudden changes in mineral constitutents and the temperature, were the controlling factors.

These studies also showed that in order to prepare the water for efficient treatment on the slow sand filters it was necessary to add certain chemicals to the river water for about one-half of the average year, these chemicals being sulphate of alumina, lime, clay, and bleaching powder, either added singly or in combination, according to the character of the river water. It was also considered advisable that all of the river water, whether chemically treated or not, should be passed through contact baffles, the same being built of concrete in tank form and divided into units of about 0.11 acre each, or $2\frac{2}{3}$ acre for a minimum capacity of 200,000,000 gallons daily, the tanks being filled with gravel ranging in size from $\frac{1}{2}$ to 1-inch, the depth of the filtering layer being about 8 feet.

It was also deemed advisable to install latitudinal baffles in the large sedimentation reservoirs in order to obtain more complete water displacement and thus increase their efficiency.

It was learned in the course of the experimental studies that prolonged plain sedimentation did not eliminate the necessity for using chemicals in the preparatory treatment for probably half the aver-

age year. Without chemical treatment even as long a period as ten days storage would not at all times suitably prepare the water for slow sand filtration.

The results of the studies of 1910–11 were put into effect, and the preliminary filters and latitudinal baffles built, the bulk of the improvements being completed in December, 1913. There now remain to be installed appliances for the introduction of the coagulating and correcting chemicals to the water. When these have been made the slow sand filters will be able to furnish a filtered water at all times satisfactory in appearance and from a hygienic standpoint.

The work of the Pittsburgh filtration plant in reducing the death rate from typhoid fever in that city has been remarkable. Whereas for the 9 years prior to 1909, without filtration, the annual typhoid fever death rate in Pittsburgh averaged 125 per 100,000, for the four years ending 1913, with filtration, the annual average was 10, a reduction of 92 per cent; a result which has never been equalled in the history of water purification.

GENERAL

The foregoing specific comments, referring to the slow sand filter plants at Lawrence, Albany, Philadelphia, Washington and Pittsburgh have been set down to show the difficulties under which filters of this type are obliged to work when called upon to treat muddy waters. It is noted that in every instance, with the sole exception of Lawrence, the original design has been improved upon, and the preparatory treatment of the water made more complete. This was found imperative very shortly after the various plants went into service. Albany has added preliminary roughing filters; Philadelphia already had roughing filters which have proved but a limited success, and the use of coagulants is being considered in connection with preparatory treatment; Washington has added the coagulation process; and Pittsburgh has built roughing filters and is preparing to use coagulants and certain other chemicals.

Of other slow sand filter plants in the United States the Wilmington, Deleware plant includes preliminary roughing filters, as does the plant at Steelton, Pennsylvania; and Springfield, Massachusetts and Indianapolis, Indiana use coagulants. At practically all slow sand filter plants the final filtered product is sterilized with hypochlorites. At Lawrence the clarity of the Merrimack River water makes un-

necessary the use of roughing filters, nor are coagulants used. The same is true at Providence, Rhode Island and New Haven, Connecticut but with these exceptions it is becoming difficult to locate a slow sand filter plant which does not in some important respect depart from the original ideas of what constituted that system of water purification, or which does not in some way make use of certain inherent ideas upon which are based the rapid sand system of water filtration.

GROWTH OF RAPID SAND FILTRATION IN THE UNITED STATES

The first municipal rapid sand filtration plant was built at Somerville, New Jersey in 1885, and between that date and January, 1914, upwards of 450 municipal filter plants of this type have been built or are now under construction in the United States. At this date the total daily capacity of filtration plants of the rapid sand type is 1,745,000,000 gallons, and a total of nearly 12,000,000 people are being supplied with water so filtered. This population is distributed as follows:

Geographic distribution of population served by rapid sand filter plants

District	*Per cent*
New England States	2.0
Middle Atlantic States	22.0
South Atlantic States	10.0
Ohio Valley	26.0
Upper Mississippi Valley	13.0
Lower Mississippi Valley and Gulf	24.0
Rocky Mountain Region	0.2
Pacific Coast	3.2

In 1890 there were very few rapid sand filtration plants in operation in this country, and the decade which followed was fruitful with results of scientific investigation into the merits of the new process. The first series of public investigations were conducted at Providence in 1893, to be followed by even more elaborate studies at Louisville, Kentucky, Pittsburgh, Pennsylvania, Cincinnati, Ohio, and Washington, District of Columbia. These investigations were in no small measure responsible for the wonderful growth of rapid sand filtration during the past 15 years, for the theory of the process was thoroughly worked out, and the whole idea becoming well understood was placed on a solid footing.

The type of construction changed abruptly about 1900, rectangular concrete tanks frequently replacing the circular wooden or steel tanks formerly used. The rectangular shape of the tanks made necessary the use of compressed air to agitate the sand layer while washing the filter, and later the application of wash water at high velocities. Both of these methods of filter cleaning had been used before, and therefore were not new in principle, but their use with rectangular shape filter tanks became more general, and supplanted the mechanical stirrers and wash water applied at low rate used in the older type of rapid sand filters, and from which they derived their original name, "Mechanical" filters.

In the following tables there is given a fairly complete list of the rapid sand filter plants built in the United States since the first one was constructed at Somerville in 1885. In the second of the two tables are listed the new plants built where no rapid filter plant existed before, and also those additions to existing plants made between 1910–1913. All plants under construction on January 1, 1914, are also included in this table.

As will be seen from the following table, in the decade 1900–10 many important rapid filter plants were built in this country, among the largest being Little Falls, New Jersey; New Orleans, Louisiana; Cincinnati, Ohio; Louisville, Kentucky and Columbus, Ohio; the respective daily capacities of these plants being 32, 44, 112, 36 and 30 million gallons. As it is beyond the scope of this paper to describe the experiences at all of the plants listed in the accompanying tables, it will be sufficient to refer in detail only to the five large plants just mentioned, which will be taken up in the order named.

Little Falls, New Jersey. This plant was completed in 1902, and was built for the East Jersey Water Company which supplies with water the cities of Paterson, Passaic, Montclair and several other communities in northeastern New Jersey. The total population supplied from the Little Falls plant is in round numbers 250,000, and the capacity of the filtration plant is 32,000,000 gallons daily.

This was the first plant built wherein the filter tanks were of rectangular concrete construction, and in numerous respects all of the rapid sand filter plants built in the past dozen years are patterned after it with respect to the scheme of layout, and automatic or easily manipulated hydraulic and electrical contrivances.

The waters of the Passaic River, which this rapid sand filter plant

Rapid sand filter plants built in the United States between 1885 and 1910

STATE	NUMBER OF PLACES	TOTAL POPULATION (1910)	TOTAL DAILY FILTERING CAPACITY
			gallons
Alabama	5	172,000	23,325,000
Arkansas	1	46,000	5,500,000
California	6	220,000	14,567,000
Colorado	2	5,000	975,000
Connecticut	2	26,000	3,500,000
Georgia	15	337,000	34,350,000
Illinois	16	321,000	44,650,000
Indiana	12	250,000	42,500,000
Iowa	12	238,000	30,750,000
Kansas	9	115,000	11,850,000
Kentucky	7	330,000	50,250,000
Louisiana	2	367,000	45,000,000
Maine	8	83,000	18,800,000
Maryland	4	15,000	2,550,000
Massachusetts	2	11,000	2,500,000
Michigan	5	45,000	8,910,000
Minnesota	6	37,000	4,528,000
Mississippi	4	55,000	7,300,000
Missouri	13	180,000	24,350,000
Nebraska	1	5,500	400,000
New Hampshire	2	6,500	1,114,000
New Jersey	19	709,000	86,600,000
New York	22	321,000	57,900,000
North Carolina	18	222,000	22,525,000
Ohio	24	1,083,000	216,420,000
Oklahoma	6	112,000	13,500,000
Oregon	3	23,000	3,500,000
Pennsylvania	45	892,000	123,725,000
Rhode Island	7	74,000	13,000,000
South Carolina	6	101,000	16,500,000
Tennessee	5	101,000	17,500,000
Texas	4	38,000	3,950,000
Vermont	1	20,000	2,000,000
Virginia	9	198,000	23,350,000
Washington	2	15,000	1,000,000
West Virginia	5	83,000	13,500,000
Wisconsin	4	65,000	7,360,000
Totals	314	6,922,000	999,999,000

Rapid sand filter plants built and building in the United States during period 1910-1913. (Date of compilation, January, 1914)

STATE	ESTIMATED POPULATION SERVED IN 1914 BY PLANTS BUILT 1910–1914	NUMBER OF PLACES			TOTAL DAILY FILTERING CAPACITY		
		Entirely new plants	Additions to old plants	Total	Entirely new plants	Additions to old plants	Total
					gallons	*gallons*	*gallons*
Alabama	5,700	0	2	2	0	4,000,000	4,000,000
California	25,000	6	1	7	2,140,000	1,000,000	3,140,000
Colorado	11,000	1	0	1	4,000,000	0	4,000,000
Connecticut	29,000	2	0	2	2,000,000	0	2,000,000
Georgia	80,000	4	3	7	3,000,000	14,000,000	17,000,000
Illinois	170,000	11	6	17	21,950,000	18,825,000	40,775,000
Indiana	43,000	2	1	3	12,000,000	1,000,000	13,000,000
Iowa	26,000	2	2	4	1,500,000	8,000,000	9,500,000
Kansas	62,000	9	2	11	9,040,000	5,775,000	14,815,000
Kentucky	22,000	2	2	4	7,000,000	2,000,000	9,000,000
Louisiana	7,100	1	0	1	500,000	0	500,000
Maine	19,000	1	2	3	1,135,000	9,500,000	10,635,000
Maryland	754,000	3	0	3	133,000,000	0	133,000,000
Massachusetts	5,000	1	0	1	500,000	0	500,000
Michigan	200,000	2	1	3	30,000,000	750,000	30,750,000
Minnesota	404,000	3	0	3	41,500,000	0	41,500,000
Missouri	947,000	6	0	6	168,300,000	0	168,300,000
Montana	4,500	2	0	2	2,325,000	0	2,325,000
New Jersey	149,000	2	2	4	30,300,000	3,000,000	33,300,000
New York	114,000	4	3	7	25,750,000	20,000,000	45,750,000
North Carolina	55,000	9	6	15	4,350,000	5,100,000	9,450,000
North Dakota	43,000	3	0	3	7,000,000	0	7,000,000
Ohio	248,000	8	4	12	31,820,000	27,500,000	59,320,000
Oklahoma	37,000	5	1	6	5,750,000	1,000,000	6,750,000
Oregon	24,000	6	0	6	3,967,000	0	3,967,000
Pennsylvania	223,000	16	6	22	44,500,000	9,825,000	54,325,000
Rhode Island	4,500	0	1	1	0	1,000,000	1,000,000
South Carolina	39,000	5	1	6	3,500,000	800,000	4,300,000
Tennessee	13,000	0	2	2	0	5,000,000	5,000,000
Texas	277,000	5	0	5	28,000,000	0	28,000,000
Virginia	1,00	1	1	2	1,000,000	250,000	1,250,000
West Virginia	17,000	3	1	4	3,000,000	500,000	3,500,000
Wisconsin	30,000	2	0	2	5,500,000	0	5,500,000
Wyoming	8,000	3	0	3	1,500,000	0	1,500,000
Totals	4,096,800	130	50	180	635,827,000	138,825,000	774,652,000

was called upon to purify are not muddy, although, as will be shown in the next table, following heavy rains the turbidity will sometimes rise to over 100 parts per million for a day or so. The river above Little Falls is not badly polluted, and the chief objection to it in its raw state is a noticeable color, due to dissolved vegetable matter coming from several large swamps situated on the drainage area. It was to remove the bulk of this color, and to guard against the effects of possible contamination, that the Water Company decided to build the filter plant, a full description of which is given in the *Transactions of the American Society of Civil Engineers*, Vol. 50, p. 394 (1903).

Records of operation of Little Falls, New Jersey, rapid filter plant

MONTH (1912)	SULPHATE OF ALUM, GRAINS PER GALLON	PER CENT WASH WATER USED	PARTS PER MILLION								BACTERIA PER CUBIC CENTIMETERS		
			Turbidity				Color				Average		
			Average		Highest Day		Average		Highest Day				Highest day in filtered water
			River	Filtered	River	Filtered	River	Filtered	River	Filtered	River water	Filtered water	
January	0.74	1.7	7	0	9	0	32	5	38	8	2,100	2	17
February	0.65	1.8	13	0	30	0	35	6	53	11	9,700	6	43
March	0.98	2.7	26	0	118	0	52	6	92	15	10,000	5	60
April	1.07	2.8	8	0	13	0	41	6	46	9	1,600	3	20
May	1.46	3.3	8	0	13	0	55	7	66	11	1,300	2	10
June	1.60	3.3	10	0	12	0	56	8	65	11	1,100	1	3
July	1.65	3.4	9	0	11	0	46	9	55	11	600	4	13
August	1.62	3.4	8	0	17	0	43	9	52	10	700	2	5
September	1.47	3.1	7	0	10	0	41	8	64	10	750	2	4
October	1.93	3.1	10	0	23	0	56	7	78	10	1,100	4	21
November	3.16	3.0	10	0	22	0	61	14	71	22	2,000	4	11
December	1.86	2.3	8	0	12	0	40	11	53	19	1,800	3	10
Annual average	1.51	2.8	10	0	...	...	46	8	...	...	2,730	3	.....

The records given in the last table show that while on one day in 1912 the turbidity of the water rose to 118 parts per million the yearly average turbidity of the raw water was only 10 parts. The filtered water was free of turbidity at all times. The color of the unfiltered water reached a maximum of 92 parts per million in March, while the average for the year was 46 parts. The filtered water color averaged 8 parts for the entire year, and here it should be noted that the aim at this plant is to reduce the color so that the filtered water

shall not contain more than 10 or 15 parts. In November and December the color of the filtered water was slightly in excess of this upper limit for short periods. The bacteria in the filtered water averaged 3 per cubic centimeter, and the highest number recorded on any one day during the year was 60 per cubic centimeter.

The average amount of sulphate of alumina used for coagulation during the year was 1.51 grains per gallon, equivalent to 216 pounds per million gallons of water treated. The amount of filtered water used for cleaning the filters averaged 2.8 per cent of all the water filtered during the year. The filters are washed by means of compressed air and wash water applied at the rate of about 8 gallons per square foot per minute.

As to the reduction of typhoid fever, in the largest city supplied by this plant, Paterson, the average death rate from typhoid fever for the 5 years prior to the filtration of the water supply was 32 per 100,000 population, and for the 10 years following, with filtration, the average death rate from this disease was 9, a reduction of 72 per cent.

New Orleans, Louisiana. Before this rapid sand filter plant was built careful investigations were conducted at this city into the relative applicability under the local conditions of the slow sand and rapid sand systems of water purification. These investigations were terminated in August 1901, and in the report prepared from the results obtained the following conclusions were drawn

Conclusions as to the relative applicability of the slow sand and rapid sand filtration systems in the purification of the water supply of New Orleans.

Three general steps in the systems for clarification and purification have been considered in various combinations, as follows:

1. Plain subsidence in basins for several days.
2. Supplementary subsidence in basins, with the aid of coagulant.
3. Filtration, either at a slow rate through sand beds, or at a rapid rate through filters provided with mechanical devices for cleaning the sand.

On account of the large amount of fine clay particles, the suspended matter is held in suspension indefinitely, and cannot be removed on the average by any practical periods of plain subsidence, even by periods of a week or more. When this subsided water is filtered at slow rates through thick layers of fine sand, a satisfactory clarification and purification results for a portion of the time. However, this system would produce muddy effluents for months at a time, and what is more of a factor, the cost of cleaning and removing the clogged sand layers would be prohibitively excessive.

Since the fine particles of suspended matter cannot be removed by plain subsidence, it is necessary to add something to the water to bring them to-

gether in aggregates or groups which of themselves would have a subsiding value great enough to cause them to settle within a reasonable period. Sulphate of alumina can be used for this purpose, and when added to the water forms a perfectly insoluble gelatinous precipitate, which, uniting with the clay, causes it to gather into flocks or aggregates which settle much more rapidly than the diffused clay particles themselves. This precipitate also has the power of attracting, enveloping, or absorbing stray particles of suspended matter, including bacteria, so that the water by this method of coagulation and supplementary subsidence can be made fairly clear. In practice, water which has been purified by plain subsidence followed by supplementary subsidence with a coagulant is, strictly speaking, never perfectly clear. This is because it is difficult to remove the last traces of turbidity by this process alone. Indeed, it is much more satisfactory to make the final step in the clarification and purification process by the use of a suitable filter.

Two kinds of filters can be used for this final clarification and purification of the effluent of the coagulating basins, namely, the English (slow sand) and American (rapid sand) filters. These filters need but little explanation in this chapter. The English (slow sand) filter, as is well known, consists of a bed of sand through which the water, without coagulation, but after being subjected to plain subsidence, filters at a slow rate; while the American (rapid sand) system treats the water after both plain subsidence and supplementary subsidence with the aid of a coagulant, and filters it at a rapid rate through a bed of sand which is intermittently freed from matter which obstructs the passage of water, by reversed currents of water, and by agitation. In the English (slow sand) filter the accumulated matter which clogs the filter is removed by draining and scraping.

Either filter would be adapted to the purification of the Mississippi River water, and the decision as to which would best suit local conditions should be dependent upon the cost per million gallons of filtered water, taking into consideration the difference in first cost. In the total first cost the American (rapid sand) system is fully 25 per cent cheaper than the modified English (slow sand) system when estimates are made on the same basis.

Taking into consideration all the available evidence, it is concluded that the American (rapid sand) system is best adapted to the purification of the Mississippi River water at New Orleans.

A rapid sand filter plant was built in accordance with the recommendations above cited, and is doing excellent work in the purification of the muddy Mississippi River water. The system consists of preliminary sedimentation basins, chemical mixing and coagulating basins, and rapid sand filters capable of filtering 40,000,000 gallons of water daily. Lime and sulphate of iron are used as coagulants, and the filters are cleaned by the application of filtered water under high velocities, no other means being provided for agitating the sand layer during washing. Summarized results from this plant for the year 1912 are contained in the following table:

Results of operation of New Orleans rapid sand filtration plant

MONTH (1912)	GRAINS PER GALLON COAGULANT		PER CENT WASH-WATER USED	TURBIDITY (PARTS PER MILLION)				BACTERIA PER CUBIC CENTIMETER			
				Maximum day		Average		Maximum day		Average	
	Lime	Iron		River	Settled	River	Settled	River	Filtered	River	Filtered
Jan.....	3.68	0.32	0.6	1,250	60	900	32	3,100	65	2,000	17
Feb.....	3.68	0.59	0.7	675	110	475	40	15,000	65	6,000	27
Mch....	3.51	0.73	0.5	1,600	75	900	35	16,000	3,200*	7,000	500*
Apr.....	3.21	0.98	0.3	1,350	150	950	26	11,000	2,300*	3,000	750*
May....	3.73	1.03	0.3	1,100	100	700	35	8,000	650	2,100	95
June....	4.60	0.25	0.3	1,400	525	625	34	9,500	190	2,800	65
July....	4.46	0.06	0.3	2,200	100	1,350	40	8,500	300	3,300	70
Aug.....	4.51	0.13	0.3	1,850	400	1,100	75	13,000	120	3,400	36
Sept....	4.65	0.11	0.3	1,000	1700	750	65	10,000	85	3,700	34
Oct.....	4.87	0.09	0.3	725	110	550	55	9,500	180	2,000	23
Nov.....	5.78	0.04	0.4	550	190	400	35	32,000	150	1,900	15
Dec.....	5.71	0.05	0.4	675	150	425	35	6,000	55	1,800	10
Annual average	4.41	0.33	0.4	.....	.....	750	40	.....	.....	3,300	40

*Abnormal, due to algae growths.

The above results show that with relatively small amounts of coagulating chemicals, and with the very thorough pretreatment of the raw Mississippi River water, the filters delivered throughout the year a water free from noticeable turbidity and containing on an average but 40 bacteria per cubic centimeter. In computing this average the results for the months of March and April are excluded for the reason that the high numbers recorded in those months were caused by algae growths, and after growths of harmless bacteria in the filtered water following the use of hypochlorite. The amount of wash water used in cleaning the filters is also worthy of note, averaging the very low figure of 0.4 per cent of the total amount of water filtered. This was due, of course, to the very complete preparatory treatment given the water prior to its application to the filters.

Before the introduction, in 1909, of the new water works system, including the filtration works, in New Orleans, the city depended for its supply almost exclusively upon cistern water caught from roofs and stored in cypress tanks above the ground. Less than 7,000 premises had connections with the public supply furnished direct

from the Mississippi River. The typhoid and malarial fevers existing in New Orleans prior to 1910 were not largely water borne, but due in the main to the spread of local foci through the agencies of flies, impure milk and other food supplies, and greatly augmented by the general lack of sewerage facilities. Since 1908 the rate at which connections have been made to the new sewer and water systems has been slow, as shown by the following statistics:

Data relative to connections made to sewer and water systems of New Orleans since 1907

BY DECEMBER 31 OF	ESTIMATED POPULATION	APPROXIMATE NUMBER OF PREMISES CONNECTED TO		APPROXIMATE PER CENT OF POPULATION CONNECTED TO	
		Sewers	New Water Works	Sewers	New Water Works
1907	318,000	925	0	1	0
1908	325,000	3,400	2,000	4	3
1909	331,000	9,000	15,800	11	16
1910	339,000	13,800	26,000	16	31
1911	347,000	20,200	38,000	22	41
1912	355,000	28,500	46,000	32	52

The above data indicate pretty conclusively why the typhoid fever death rate has not decreased as sharply as in some other cities, following the installation of water filtration works. But that there has been a sharp diminution in typhoid and malarial fever in New Orleans since the municipal sanitary improvements made, is shown by the statistics given in the following table, compiled from U. S. Census reports. That the reductions noted should have been so great is remarkable in view of the fact that in 1912 but one-third of the population had connected to the sewerage system, and but one-half the population were being supplied with pure filtered water.

Death rates in New Orleans from typhoid and malarial fever per 100,000 population

DATE INCLUSIVE	MALARIAL FEVER	TYPHOID FEVER	PER CENT REDUCTION COMPARED WITH PERIOD 1881-1910	
			Malarial Fever	Typhoid Fever
1881–1910	93	33	..	..
1911	9	31	90	9
1912	8	14	91	58

Cincinnati, Ohio. This rapid sand filter plant is the largest of its type in operation in this country, although larger rapid sand filters are now under construction at St. Louis, Missouri (160,000,000 gallons) and Baltimore, Maryland (128,000,000 gallons). It has a daily filtering capacity of 112,000,000 gallons and was completed in 1907.

In 1896 a special Engineer Commission, investigating conditions for the extension and betterment of the water supply system of Cincinnati, recommended that the Ohio River water be clarified by several days' subsidence and afterwards filtered through slow sand filters. The Commission of Engineers of 1897, however, decided that before proceeding with the filtration project reliable data should be obtained with reference to exact local conditions. Accordingly, an experimental plant was built, consisting of several sedimentation tanks and some fifteen slow sand filters, and the performances of these devices were carefully noted during the year 1898. In the autumn of that year it became apparent that the plan proposed by the Commission of 1896 was of very doubtful expediency, and a series of comparative tests was then begun to show the relative efficiency under the local conditions of the modified slow sand, and rapid sand filtration systems, the former system differing from the usual slow sand process only in that the water was clarified by coagulation and sedimentation before filtration. In January 1899 the following conclusions were reached:

With respect to the slow sand filtration system

In the absence of heavy and long continued freshets in the river, three days' subsidence would be adequate to allow the local water to be satisfactorily clarified and purified by this type of filter (slow sand) and at a reasonable cost. When heavy and long continued rises in the river do occur, the method under consideration would not be a success, with filters constructed within conventional limits as to thickness of sand layer and operated within conventional limits as to rate of filtration. Their failure under such conditions would be due to the excessive turbidity of the effluent; the diminution in bacterial efficiency toward the end of long rises; and to the abnormal expense of cleaning the sand layers.

It would be possible by increasing sufficiently the thickness of the sand layer or by decreasing sufficiently the rate of filtration, or by a combination of these two modifications, to obtain uniformly an effluent of satisfactory character and appearance from the local river water after plain subsidence for three days. But the cost of such procedures would clearly make them prohibitive.

To make use successfully at all times of English (slow sand) filters it would be necessary at times of freshets to give the Ohio River at Cincinnati further preparatory treatment than is afforded by three days of plain subsidence.

With respect to the modified slow sand filtration system

The evidence shows that the modified English (slow sand) system of purification is applicable to the Ohio River water at Cincinnati, based on the satisfactory clarification and purification of the water. It would, however, be very much more difficult to operate, and somewhat more expensive, than is understood to be the case with those purification works of the English (slow sand) method now in use at various cities.

With respect to the rapid sand filtration system

The evidence obtained during these investigations shows that it is practicable to clarify and purifiy the Ohio River water in a satisfactory manner by either the modified English (slow sand) system or by the American (rapid sand) system. Of these two systems, the experience and data indicate clearly that the American (rapid sand) system would be the less difficult to operate; would be somewhat cheaper; would give substantially the same satisfactory quality of filtered water; and could be much more readily and cheaply enlarged for future requirement. It is therefore considered that the American (rapid sand) system of clarification and purification would be the more advantageous to adopt for the local water supply.

The raw Ohio River water is first allowed to settle for some three days in a large reservoir, coagulant being then added and a period of from two to three hours allowed for coagulation and supplementary sedimentation to take place. The water is then applied to the rapid sand filters, of which there are twenty-eight 4,000,000 gallon units. The filter tanks are rectangular in plan and built of concrete. The filters and all appurtenances are contained within a brick building with a concrete roof. The filtered water basin is located outside the filter building, and has a capacity equal to about two hours supply when the plant is working under full load.

Results of operation of the Cincinnati rapid sand filtration plant, for the year 1910

Average gallons filtered per day		48,500,000
Grains per gallon	Lime	0.84
	Iron	1.79
Per cent wash water used		3.5
Turbidity parts per million—Average for Year	River	190
	Plain settled	80
	Applied	21
	Filtered water	0
Bacteria per cubic centimeter—Average for year	River	8900
	Plain settled	3200
	Applied to filters	750
	Filtered water	75

The figures given in the last table show that the Cincinnati rapid sand filter plant furnishes a filtered water free from turbidity, and containing less than one per cent of the bacteria originally present in the untreated Ohio River water.

As to the reduction in typhoid fever, the experience in Cincinnati has been very gratifying. The purification works went into service in 1908, and for eight years prior to that time the average death rate from the disease averaged 55 per 100,000 population. During the succeeding five years these rates fell to 19, 13, 6, 11 and 6 respectively per 100,000 an average of 11 for the period, or a reduction of 82 per cent.

Louisville, Kentucky. This rapid sand filtration plant, having a daily filtering capacity of 36,000,000 gallons, was placed in operation late in 1909. The raw Ohio River water is subjected to plain sedimentation in the old Crescent Hill reservoir, after which coagulant is added, a period of coagulation and supplementary sedimentation allowed, and the partially clarified water then applied to rapid sand filters.

As early as 1884 the filtration question was investigated at Louisville, but those earlier studies were confined to slow sand filtration. On account of the frequently muddy character of the Ohio River water at Louisville, these investigations showed that slow sand filtration could not efficiently and economically purify it.

In 1895 an exhaustive series of investigations into the merits of the rapid sand filtration process was begun. Numerous proprietary systems were studied, and it was largely the result of this work that placed the rapid sand filtration process intelligently before the world, and was in no small measure responsible for the rapid growth of the process from the year 1900 on.

The Louisville investigations were continued practically without interruption for two years, during which the conclusion was reached that it was entirely feasible to thoroughly clarify and otherwise purify the Ohio River water by plain sedimentation, coagulation and filtration through rapid sand filters.

The rapid sand filter plant which was later built has now been in successful operation for some five years. The year 1911 was a representative one, and the results of operation of the filtration works for that year are given in the following table:

Average results of operation of the Louisville rapid sand filtration plant for the year 1911

Daily volume of water filtered (gallons)		24,000,000
Sulphate of alumina used (grains per gallon)		1.73
Per cent wash water used		2.37
Turbidity (parts per million)	River water	225
	Filtered water	0
Bacteria (per cubic centimeter)	River water	10,300
	Filtered water	135

From the foregoing results it is seen that the filters produce from the muddy Ohio River water an effluent free from turbidity and containing but 1.3 per cent of the bacteria originally present in the untreated water. The volume of wash water used in cleaning the filters averaged 2.37 per cent of the total volume of water filtered.

As to the reduction of typhoid fever in Louisville the results are gratifying, showing a steady annual decrease since the filter plant went into service. For the 10 years ending 1909, and prior to the filtration of the water, the average death rate from typhoid fever in Louisville was 56; and during the three succeeding years it was 31, 25 and 19, respectively, an average of 25, or a reduction of 55 per cent.

Columbus, Ohio. This purification plant, put into operation in 1908, is designed to soften the very hard Scioto River water as well as to clarify and otherwise purify it. The works have a capacity of 30,000,000 gallons, the tanks and basins are built of reinforced concrete, and the buildings are of brick with tile roofs.

The plant comprises main reaction chambers, holding about one hour's flow as a minimum and wherein the bulk of the softening action takes place, coagulating basins, 12 in number, and allowing a period of about ten hours for coagulation and sedimentation, and 10 rapid sand filter units, each having a daily filtering capacity of 3,000,000 gallons. The filtered water basin, also built of concrete, is located underneath the filters, and has a capacity of 10,000,000 gallons.

The water which this plant is called upon to purify is not only frequently muddy, but is one of the hardest waters in the country. As delivered from the purification plant it is relatively soft and free from bacterial life, clear and palatable. The results of operation of this plant during a representative year are given in the following table:

Results of operation of the Columbus rapid sand filtration and softening plant for the year 1911

1911			HIGHEST DAY	AVERAGE FOR YEAR
Volume of water filtered daily. (Gallons)			19,700,000	14,900,000
Per cent of wash water used				1.3
Coagulating and softening chemicals used (grains per gallon)		Lime		7.5
		Soda ash		4.3
		Sulphate of alumina		1.57
Parts per million	Color	River water	75	28
		Filtered water	17	7
	Turbidity	River water	733	68
		Filtered water	0	0
	Total Hardness	River water	321	245
		Filtered water	110	84
	Alkalinity	River water	203	150
		Filtered water	73	45
	Incrustants	River water	131	45
		Filtered water	60	38
Bacteria per cubic centimeter		River water	220,000	11,470
		Filtered water	2,800*	55

*Abnormal. Caused by growths of harmless bacteria.

From the above results it is seen that the total hardness of the Scioto River was reduced on the average to less than 100 parts per million, a reduction of over 66 per cent., and the incrusting constituents to 38 parts, a reduction of 60 per cent. The bacteria in the raw water were reduced by 99.5 per cent on an average.

The reduction of typhoid fever in Columbus since the filtration works were put into operation in 1908 has been most gratifying. For the nine years prior to filtration of the water supply the average death rate from typhoid fever was 62; and for the four years following 1908 it was 17, 17, 14 and 19, respectively, averaging 17, and equivalent to a reduction of 73 per cent.

REASONS FOR WATER COAGULATION AND EFFECT THEREOF ON THE CHARACTER OF FILTER EFFLUENTS

One of the earliest objections of the rapid sand filtration process of water purification was the fact that the water was treated with a chemical in the preparation thereof for filtration. The layman, and indeed some sanitarians and many physicians, viewed with appre-

hension the use of alum and iron salts in connection with the purification of water, believing that some of the chemical passed through into the final effluent to the detriment of the person drinking it. Indeed, some hold this view today, even in the face of the fact that the claim was scientifically and practically exploded many years ago. It has also been claimed that the use of certain chemicals made the filtered water less desirable for steaming purposes, as, for example see the conclusions of the Pittsburgh Filtration Commission on page 40 of this paper.

Of late years there has been considerable discussion of "red water" troubles, that is, water to which a reddish brown color is imparted by suspended particles of iron oxide. It has been claimed by some that the bulk of complaints from red water troubles come from cities where the rapid sand filter process is used. It may be well, therefore, to dwell at some length upon the reasons why coagulation is an important adjunct to water filtration, and the actual effect which the addition of coagulating chemicals has upon the physical quality of filtered water.

DEFINITION OF COAGULATION

By coagulation is meant the use of certain harmless chemicals which, upon being added in minute quantities to water, produce a gelatinous precipitate that envelopes the mud, clay, organic matter and bacteria, and causes them to form into aggregates of such size that they can be much more easily removed, either by sedimentation or by filtration, than is the case with uncoagulated water. This treatment makes possible the adequate preparation of muddy water for final filtration, without the construction of large sedimentation reservoirs, wherein the removal of the mud in the water takes place very slowly and more or less incompletely.

COAGULATING CHEMICALS

The chemicals most commonly used for the coagulation of water are compounds of aluminum and iron, and of these potash alum, sulphate of alumina, and sulphate of iron are the most extensively employed.

The manufacture of alum is of great antiquity, and for many centuries this chemical has been used for coagulating water, as an aid to speedy clarification. The manufacture of alumina sulphate from

bauxite and limeclay is of more recent origin. The sulphate of iron used in water coagulation is, for the most part, a byproduct of iron and steel industries.

The choice between the different coagulating chemicals is partly based upon their efficiency as coagulants, and this refers directly to the percentage of available alumina or iron which they contain. In a general way it may be said that potash alum, and sulphate of alumina cost about 1 cent a pound, while sulphate of iron costs about ½ cent a pound. In this country sulphates of alumina and iron are the most widely employed in water purification.

In composition these chemicals show considerable variation, but they may be bought on a basis of a guaranteed percentage of available alumina or iron. The essential feature is that the chemical shall be basic, namely, that it shall contain more available alumina or iron than is required to combine with the dissolved sulphuric acid, and that it shall contain no free sulphuric acid. The approximate composition of these chemicals is as follows:

Approximate percentage composition of coagulating chemicals

CONSTITUENT	PURE POTASH ALUM	SULPHATE OF ALUMINA	SULPHATE OF IRON
Matter insoluble in water	.....	0.30	0.50
Alumina (Al_2O_3)	10.77	17.00	.....
Iron (Fe_2O_3 and FeO)	.....	0.25	57.50
Potash (K_2O)	9.93	.....	.....
Sulphur trioxide (SO_3)	33.76	38.70	28.80
Water (H_2O)	45.54	43.75	13.20

ACTION WHICH TAKES PLACE UPON THE ADDITION OF COAGULATING CHEMICALS TO WATER

When potash alum or sulphate of alumina is applied to water, the chemical is rapidly and completely decomposed by the alkaline compounds naturally present in the water. Ordinarily this alkalinity is due to carbonates and bicarbonates of lime and magnesia. The sulphuric acid portion of the coagulating chemical displaces the weak carbonic acid of the alkaline compounds above mentioned. As a result soluble sulphates of lime and magnesia are formed and equivalent amounts of carbonic acid and alumina are liberated. The latter unites with the water and forms the white, insoluble and gelat-

inous precipitate, known as aluminum hydrate, and which has the property of massing together various impurities as mentioned above.

The amount of coagulating chemical that it is necessary to use depends upon the character of the water to be treated, especially the turbidity or color of such water. Very turbid or very highly colored waters frequently require several grains of coagulating chemical per gallon of water. At most plants, however, the annual average is not greatly in excess of 1 grain per gallon. This quantity is equivalent to about 17 parts per million by weight, or 143 pounds per million gallons of water treated.

One grain per gallon of sulphate of alumina requires for its decomposition about 7 parts per million of alkalinity, depending upon the precise strength of the chemical used. This means, as explained above, that 7 parts per million of the carbonates and bicarbonates of lime and magnesia naturally present in the untreated water are converted into the sulphates of lime and magnesia.

The total hardness of the water is unaffected by the application of coagulating chemicals, such as sulphate of alumina, when decomposed by the natural alkaline compounds of the water to be treated.

The amount of carbonic acid which is liberated for each grain of applied sulphate of alumina is equal to about 3.5 parts per million, roughly one-fifth of a grain per gallon.

Under some circumstances the liberation of carbonic acid seems to increase somewhat the rate at which clear filtered and oxygenated water corrodes certain forms of uncoated (unprotected) metal. It is not a factor to be regarded with apprehension because the conditions are no more conducive to corrosion than are found in some of the best ground water supplies of the country. If thought advisable, this carbonic acid may be removed by the addition of lime.

The average consumer cannot ordinarily detect any effect due to the addition of coagulating chemicals as they are bound to be decomposed in order to become effective. Neither can the ordinary consumer detect any changes as regards hardness, because the total hardness of the water is unchanged.

Where water is boiled or is used for steam-raising purposes, a water treated with a coagulating chemical is a little less satisfactory, but the effect is merely nominal when consideration is given to the large diminution in the accumulations formed within the boiler due to the elimination of mud where treated water is used.

As a general proposition all surface waters naturally contain a

sufficient quantity of alkalinity to completely decompose all of the chemical which is applied for coagulating purposes. In some waters, however, the natural alkalinity is so low, particularly at times of floods, that it is necessary to make up the deficiency by applying soda ash or lime to the water.

Sulphate of iron, known commercially as copperas, is the ordinary commercial byproduct in iron and steel industries. A somewhat higher grade of sulphate of iron is manufactured by a vacuum crystalizing process.

The use of sulphate of iron in water purification ordinarily requires for the precipitation of the iron the addition of lime. When added to a natural water, copperas is decomposed in a manner somewhat similar to alum, except that the resulting bicarbonate of iron is partly soluble and more or less granular. By adding lime bicarbonate of iron is changed to the gelatinous ferrous hydrate, which in turn is oxidized into ferric hydrate. The latter is insoluble and gelatinous, serving well in the massing together of impurities. To obtain satisfactory results from the use of lime and iron as coagulants it is necessary to add sufficient lime to neutralize and precipitate the iron. This must be done carefully. The use of too little lime results in poor coagulation, caused by the incomplete precipitation of the iron, some of which is usually left in solution, appearing in the effluent of the coagulating basin. The use of too much lime results in the formation of lime incrustants, which are liable to cause trouble through deposition in the sand bed and in pipes and valves.

COAGULATION IN THE PREPARATORY TREATMENT OF WATER FOR FILTRATION

Where waters in their raw state are normally clear and colorless, as is true in some parts of northeastern United States, practically no preparatory treatment is required before filtration. It is in connection with such waters as these, which more nearly approach in character the waters of western Europe, that purification by slow sand filtration, whose birthplace was in England, may be successfully practiced.

Waters which are comparatively clear but highly stained by decaying vegetation and which require treatment for the removal of bacterial life, may be purified by slow sand filtration. Such treatment however, will remove only a relatively small part of the color

dissolved in the water, and if it is desired to remove all of this color a coagulating chemical must be used. To use coagulants effectively and economically in connection with sand filters the period of preliminary coagulation and sedimentation should be at least 18 hours, and preferably 24 hours.

Many waters in the South and Central West are almost always more or less muddy. The waters of the Missouri, Mississippi and lower Ohio Rivers are good examples of this class of water. The suspended matters which cause these waters to be muddy are largely mineral, but vary greatly in the size of the particles. Some are comparatively coarse and settle out readily when the water is allowed to stand. Others are of exceeding fineness, many of them less than $\frac{1}{100,000}$ of an inch in average diameter, which is smaller than the ordinary bacterium. Turbid waters such as these, applied to filters without preliminary treatment, cannot be satisfactorily purified to the degree now demanded in this country.

It is in the purification of these muddy waters that some of the most difficult problems are found. The turbidity of many river waters shows abrupt changes from that of comparative clearness following long periods of drought, when the amount of suspended matter it contains may be less than 50 parts per million (430 pounds of dry mud per million gallons) to periods of great muddiness during freshets, when it may contain as much as 2000 or more parts per million (17,000 pounds or more per million gallons). At Pittsburgh the highest amount of suspended matter recorded in 1909 was 1300 parts per million, equal to 11,000 pounds of dry mud per million gallons. Changes from comparatively clear to very muddy water occur very suddenly at times, and the character of the suspended particles is subject to great variation.

To remove the great bulk of this suspended matter prior to filtration and to do it economically is not a simple problem. If it is done by plain sedimentation, then the basins in which subsidence takes place must be large enough to deal satisfactorily with the water when in its worst condition. If sedimentation is to be aided by preliminary coagulation, then the basins must be large enough to permit of adequate subsidence of the coagulated matters before the water reaches the filters. Otherwise the surface of the sand becomes clogged too quickly, requiring cleaning at prohibitively frequent intervals. Well baffled basins should have a capacity of eighteen to twenty-four hours' flow, and if the basins are not well baffled their

required capacity may be several days' flow in order to provide satisfactory results.

The practice of clarifying muddy water by coagulation with compounds of alumina followed by sedimentation probably originated in China thousands of years ago. It is certain that for many centuries it has been the practice in that country to treat tubs of turbid water with alum by inserting a crystal of the chemical in the split end of a stick and afterwards working the crystal up and down through the water until enough of the chemical has been dissolved to effect a satisfactory coagulation. It was used in Egypt for many centuries prior to its use at numerous plantations in the southern portions of the United States as an aid to the clarification of highly colored or very muddy waters. Coagulants have not had an extensive use in western Europe, owing to the comparative absence there of muddy or highly colored waters, but are now being used in quite a number of sand filtration plants in Belgium, Holland, and in Northern Germany. Among the larger cities may be mentioned Antwerp, Schiedam, Bremen and Hamburg.

The use of coagulants was not looked upon with great favor some ten or fifteen years ago either in this country or in Europe. Careful scientific investigation into the merits of their use at ten or a dozen places during the past fifteen years, particularly at Louisville, Pittsburgh, Cincinnati, Washington and New Orleans, and the complete absence of ill effects upon the health of communities using water regularly treated in this manner, have largely eliminated this feature from the public mind. Coagulating processes are now recognized as not only necessary for the treatment of certain classes of waters, but as permissible for use with perfect safety. The process has been carefully studied by the Royal Prussian Testing Station, and its merits now have an accredited standing in the majority of European countries.

At Cincinnati and New Orleans careful investigation with a view of ascertaining the feasibility of using coagulants in connection with the clarification of water passed through quite large settling basins before application to sand filters showed the process to be entirely feasible, although its expense was substantially greater than for mechanical filters. It was found necessary to have a settling capacity following the application of the coagulating chemical equal at least to an average flow of 18 hours and preferably 24 hours, otherwise some of the gelatinous precipitate would remain in the water as it

entered the sand filters and would clog the layer immediately at the surface so frequently as to make the cost of cleaning burdensome and abnormally expensive, as already stated.

The use of coagulants in connection with the preparatory treatment of water prior to its application to slow sand filters is quite successfully practiced in this country at Springfield, Massachusetts; Ferncliff and Poughkeepsie, New York; Indianapolis, Indiana; Washington District of Columbia and elsewhere.

Exclusive of filters of any kind and with sedimentation alone, coagulation is used for the clarification and purification of quite a number of water supplies of considerable size in the United States, among which may be mentioned Omaha, Nebraska; Leavenworth, Kansas; Kansas City, Missouri; and Nashville, Tennessee. In connection with rapid sand filter plants coagulants are always used, and at the present time there are some 450 such works in actual operation.

In the regular slow sand filtration processes, as practiced at Lawrence, Massachusetts, New Haven, Connecticut, and a few other smaller places, no coagulants are made use of, and there should not be caused by such filtration any change in the hardness of the water. In slow sand filtration as practiced at Washington, District of Columbia, Indianapolis, Indiana, and other places, where coagulants are used in the preparation of the water for slow sand filtration the effect of the added coagulant is, or should be, the same as that noted in rapid sand filtration processes. In any event the total hardness, and hence the soap consuming quality of the water, is neither increased or decreased, but the incrustants are slightly increased. Such increase in incrustants could hardly be detected by steam raisers, judged by incrustations in steam boilers. In fact it is decidedly probable that the elimination of the fine clay and silt obtained by coagulation would make the filtered water a more desirable boiler water than unclarified water.

The following results are given as illustrative of the actual changes in hardness which take place at some of the large filtration plants.

Hardness of river waters at various places before and after coagulation and filtration

CITY	AVERAGE FOR YEAR	KIND OF FILTERS	AMOUNT OF COAGULANT USED GRAINS PER GALLON	PARTS PER MILLION				INCREASE OF INCRUSTANTS IN FILTRATED WATER DUE TO USE OF COAGULANTS
				Total Hardness		Incrustants		
				River	Filtered	River	Filtered	
Springfield, Mass...	1912	Slow sand	(d) 0.24	11	11	2	4	2
Little Falls, N. J...	1903	Rapid sand	(d) 1.38	31	30	7	14	7
Louisville, Ky.....	1912	Rapid sand	(d) 1.73	95	91	29	38	9
Cincinnati, Ohio....	1910	Rapid sand	(a) 0.84 (b) 1.79	76	89	32	41	9
New Orleans, La...	1912	Rapid sand	(a) 4.41 (b) 0.33	111	60	21	25	4
Columbus, Ohio....	1910	Rapid sand	(a) 7.5 (c) 4.3 (d) 1.57	270	85	111	35	Softened

(a) Lime; (b) Iron; (c) Soda Ash; (d) Sulphate of alumina.

The above results show practically all there is to the proposition that coagulation seriously affects the water for steam raising purposes. The data show positively that this cannot possibly be so. The only detrimental change which takes place is the formation in the water of some 6 or 8 parts per million of incrustants for every grain per gallon of coagulant used. Such small increase could not be measured detrimentally by the steam user, and the undesirable features surrounding their presence in the filtered water are more than offset by the advantages inherent in a boiler water which is free from silt and clay.

THE RED WATER QUESTION

Although the entire question of corrosion of metals by water has been given considerable attention by chemists and engineers ever since the general introduction of public water supplies in this country, of late years there has been considerable active agitation of this proposition and a tendency has been shown to attach a large measure of responsibility for the appearance of iron rust in water to alleged increases in the corrosive properties of water caused by the use of certain chemicals used in the course of its purification. Thus more or

less recently there has been a great deal of discussion of the so-called "red water plague."

It is a well established fact that pure iron is in some considerable measure soluble in pure water; also that the presence of oxygen enhances the corrosive action of water on iron. The oxygen of course oxidizes the iron dissolved in water and forms iron rust, and this imparts a reddish color to the water.

All surface waters, in the absence of gross pollution or complete stagnation, contain some dissolved oxygen, in amounts ranging up to 8 or 10 parts per million. In ground waters the amount of dissolved oxygen is generally less than in surface waters. Pure soft waters will corrode metal pipes, as will even pure rain water. Neutral salts in water will also cause corrosion, but are of considerable significance from the standpoint of the aid they offer in the formation of protective coatings in pipes.

Free carbonic acid increases the corrosive action of water on metals, and is usually present in all ground waters and in the majority of surface waters. The ground water supplies of Lowell, Framingham and Malden, Massachusetts, contain from 7 to 25 parts per million of carbonic acid; and the surface supplies of other cities in the same state contain as high as 15 parts at times. At Pittsburgh, in 1912, the free carbonic acid in the Allegheny River water was at times as high as 18 parts per million, and the average for the year was about 7 parts. At Springfield, Massachusetts, in 1912, at the West Parish filter plant, the river water contained from 2 to 3 parts. The Scioto River water at Columbus, Ohio, contains about the same amount. At Little Falls, New Jersey, the colored Passaic River water in 1912 contained on an average 8 parts per million of free carbonic acid. On some days it contained as much as 15 parts.

So far as filtration processes are concerned, where slow sand filters are used without the aid of a coagulant, and where the water contains substantial quantities of carbonaceous organic matter, the bacterial processes active in these filters cause an oxidation of such matter. An increase in the amount of carbonic acid in the water is the result. In the case of rapid sand filters, or slow sand filters, where sulphates of iron or alumina are used in the preparation of the water for filtration and where lime or soda are not also used, there is an increase in the amount of carbonic acid to the extent of some 3 to 4 parts per million for each grain of coagulant used per gallon of water treated. At Little Falls, New Jersey, in 1912, the river water contained on an

average 8 parts per million of free carbonic acid, and due to the use of 1.5 grains of sulphate of alumina per gallon of water the free carbonic acid in the filtered water as it left the plant averaged 15 parts for the year, an increase of something less than 5 parts per million for each grain of sulphate of alumina used per gallon of water treated.

Within the limits of ordinary practice in the use of coagulating chemicals in water purification in this country, the assumption that the increase in free carbonic acid caused by such chemical treatment is responsible for the more or less common run of red water troubles cannot be substantiated. Such troubles are encountered in cities having pure ground water supplies, as well as in cities using the old style slow sand filtration. The cause is not one common to all cases or localities, but is due as much, and in many instances more, to the natural character of the water supply itself, regardless of the manner in which it is purified.

RELATIVE COST OF SLOW SAND AND RAPID SAND FILTRATION

Construction. In discussing the cost of building water filtration works of the slow sand and rapid sand types, respectively, consideration will be given only to those items referring to the filter plant proper. Cost of land, pumping machinery, outside connecting piping, intakes etc., in fact everything outside the filtration plant proper, will not be considered.

For slow sand filter costs the items will include the necessary filter buildings and filters with all appurtenances, all inside piping, sand handling apparatus, preliminary sedimentation basins, preliminary filters and appurtenances and clear water reservoirs.

For rapid sand filter costs the items will include the filter buildings and filters with all appurtenances, all inside piping, filter washing apparatus, coagulating and clear water basins. Thus a fairly good idea may be had of the relative cost of building purification plants of the two types.

It is true that, on account of the much greater area required, the cost for land is far greater in the case of slow sand filtration systems than for rapid sand systems. Roughly, other things being equal, land will cost twenty times as much for a slow sand filter installation as for a rapid sand plant. Furthermore, in large projects, it is often difficult conveniently to locate a site for slow sand filters, while for a rapid sand filter plant it is a relatively easy matter as a rule. If

it is necessary to go a long distance in locating an extensive and suitable area of land for a slow sand filter site there is incurred a large expense for a conduit to bring the filtered water to the city. This is very rarely necessary in the case of rapid sand filter projects. So that, in studying the comparative figures which follow, it must distinctly be borne in mind that the costs given for slow sand filter installations are really low, since the important considerations just mentioned are not charged against them.

Cost of construction of slow sand and rapid sand water filtration plants

CITY	KIND OF FILTERS	PRESENT DAILY FILTERING CAPACITY	APPROXIMATE COST PER MILLION GALLONS DAILY CAPACITY
Albany, N. Y.	Slow sand	20,000,000	$20,000 (a)
Pittsburgh, Pa.	Slow sand	200,000,000	26,000 (a)
Philadelphia, Pa.:			
Torresdale	Slow sand	250,000,000	37,700 (a)
Upper Roxborough	Slow sand	28,000,000	29,800
Lower Roxborough	Slow sand	17,000,000	26,300 (a)
Belmont	Slow sand	60,000,000	45,200 (a)
Washington, D. C.	Slow sand	100,000,000	30,000 (b)
Cincinnati, Ohio	Rapid sand	112,000,000	11,400 (c)
Columbus, Ohio	Rapid sand	30,000,000	13,000 (d)
Dallas, Texas	Rapid sand	15,000,000	13,000
Harrisburg, Pa.	Rapid sand	16,000,000	10,300
Little Falls, N. J.	Rapid sand	32,000,000	15,000
Lorain, Ohio	Rapid sand	6,000,000	14,000
New Milford, N. J.	Rapid sand	24,000,000	11,000
Watertown, N. Y.	Rapid sand	8,000,000	11,250
Weighted averages	Slow sand	.....	$32,600
	Rapid sand	.....	12,100

(a) Cost of preliminary filters included.

(b) Cost of Dalecarlia Reservoir not included. Cost of McMillan Park Reservoir included, and also cost of remodeling Georgetown Reservoir, as well as cost of coagulating basin.

(c) Cost of large plain sedimentation basin not included.

(d) Cost of softening works not included.

The above figures show that the approximate relative cost of building the slow sand and rapid sand filter plants mentioned was $32,600 and $12,100, respectively, per million gallons daily capacity. At 5 per cent the fixed charges on these sums would amount to $4.47 and $1.66, respectively, per million gallons of water filtered.

OPERATION AND MAINTENANCE

The cost of operation and maintenance of filtration plants in a large measure, varies, of course, with the quality of the raw water. In a general way the following examples will serve to show the charges ordinarily made against the operation and maintenance of representative water filter plants in this country.

Cost of operation and maintenance of slow sand and rapid sand filtration plants

YEAR	CITY	KIND OF FILTERS	AVERAGE VOLUME OF WATER FILTERED DAILY	COST OF OPERATION AND MAINTENANCE PER MILLION GALLONS OF WATER FILTERED
			gallons	
1911	Albany, N. Y.	Slow sand	20,000,000	$2.50
1912	Pittsburgh, Pa.	Slow sand	100,000,000	3.41
1911	Philadelphia, Pa.	Slow sand (a)	9,000,000	5.62
1911	Philadelphia, Pa.	Slow sand (b)	13,000,000	3.59
1911	Philadelphia, Pa.	Slow sand (c)	38,000,000	3.88
1911	Philadelphia, Pa.	Slow sand (d)	202,000,000	1.91
1912	Washington, D. C.	Slow sand	62,000,000	4.01
1912	Cincinnati, Ohio	Rapid sand	50,000,000	4.12
1911	Harrisburg, Pa.	Rapid sand	9,000,000	3.93
1912	Little Falls, N. J.	Rapid sand	30,000,000	3.20
1912	Louisville, Ky.	Rapid sand	25,000,000	3.48
1912	New Orleans, La.	Rapid sand	16,000,000	6.32
Weighted Average		Slow sand		$2.86
		Rapid sand		4.04

(a) Lower Roxborough; (b) Upper Roxborough; (c) Belmont; (d) Torresdale.

To summarize, the average cost of building seven of the largest and most modern slow sand filter plants was $32,600 per million gallons daily capacity; and, likewise, the average cost of building six of the largest, and two medium size, rapid sand filtration plants was $12,100 per million gallons daily capacity. The average cost of operation and maintenance varied widely, of course, but averaged $2.86 and $4.04 per million gallons of water filtered by the slow sand and rapid

sand systems, respectively. Adding these last figures to the fixed charge on the first cost of construction makes up the following totals:

Slow sand filtration.....$7.33 per million gallons
Rapid sand filtration..... 5.70 per million gallons

RELATIVE HYGIENIC EFFICIENCY OF SLOW SAND AND RAPID SAND FILTERS

In former years the slow sand process of water purification was favored by the majority of sanitarians and engineers because it was considered that, as compared with the rapid sand process, the former process was more nearly a "natural" one and hence less liable to failure. The actual results obtained from both systems have long since shown this assumption to be unfounded. Both processes require careful and intelligent management, but there is no room for doubt that if there is any choice between the two as regards hygienic efficiency it belongs to the rapid sand process. Well designed and built plants of this type not only can purify water of any character, turbid, colored or clear, so that the filtered product will always be clear and colorless, but are less liable to show sharp diminution in bacterial (hygienic) efficiency in cold winter months, or when the character of the raw water is seriously contaminated with certain industrial wastes. Chemical treatment is an integral part of all rapid sand filter processes, but is a makeshift when used in conjunction with slow sand filter processes; and the more complicated the chemical treatment prior to filtration the more likely are the final slow sand filters to fail.

In brief, wherever, chemicals are or should be used in the preparation of water for filtration, it is proof that the slow sand filter is out of its element and in a field which, on grounds of economy at least, belongs exclusively to the rapid sand system.

In support of the assertion that rapid sand systems are at least the equal of slow sand systems with respect to hygienic efficiency, the following results are presented showing the typhoid fever death rate in certain American cities using slow sand or rapid sand filters. Here it is seen that the residual typhoid in those cities having rapid sand filters is 27 percent less than in those having slow sand filters. This is the best evidence available of the hygienic superiority of rapid sand filtration.

Typhoid fever death rates before and after purification of water supply by slow sand or rapid sand filters

CITY	KIND OF FILTRATION	NUMBER OF YEARS AVERAGED		TYPHOID FEVER DEATH RATE PER 100,000 POPULATION	
		Before Filtration	After Filtration	Before Filtration	After Filtration
Lawrence, Mass	Slow sand	20	20	109	23
Albany, N. Y.	Slow sand	10	13	90	21
Washington, D. C.	Slow sand	6	6	57	31
Philadelphia, Pa.	Slow sand	8	5	54	22
Pittsburg, Pa.	Slow sand	9	4	125	10
New Haven, Conn.	Slow sand	7	6	46	25
Indianapolis, Ind.	Slow sand	7	6	85	25
Paterson, N. J.	Rapid sand	5	10	32	9
New Orleans, La.	Rapid sand	10	3	38	25
Cincinnati, O.	Rapid sand	8	5	55	11
Louisville, Ky.	Rapid sand	10	3	56	25
Columbus, O.	Rapid sand	9	4	62	17
Scranton, Pa.	Rapid sand	12	2	20	9
Average	Slow sand	9	9	81	22
	Rapid sand	9	5	44	16

COMPARATIVE GROWTH OF RAPID AND SLOW SAND FILTRATION IN THE UNITED STATES

The growth of water filtration in the United States, particularly during the last dozen years or so, has been remarkable. In 1900 but 1,860,000 people were being supplied with filtered water, and in 1905 this country was inferior to Japan in this regard. Since 1900 the population so supplied has increased by 830 per cent.

In the decade 1900–1910 slow sand filtration showed a remarkable increase with respect to the population supplied from such plants. This was largely due to the construction of the plants in Philadelphia, Pittsburgh and Washington, these three cities contributing over 2,000,000 of the increased population served by that system of filtration noted in the decade 1900–1910, namely 3,523,000. The increase during the same period in the number of people supplied with water from rapid sand filter plants was even more remarkable, totaling 5,422,000, or 54 per cent greater than in the case of the slow sand filter systems.

Since 1910 the slow sand filter has failed to maintain the rate of increase noted during the previous decade, the additional population served at this date, as compared with 1910, being 1,515,000. During the same period the additional population served by rapid sand filters was 4,971,000. The proof is plain, therefore, that the slow sand filter has about reached its limit, while the rapid sand filter is growing faster each succeeding year.

The following tables and diagrams will serve to show how the practice of water filtration has grown in this country, and the respective parts which slow sand and rapid sand filter processes have played in the development of this important branch of municipal sanitation.

Population of the United States supplied with filtered water at different dates

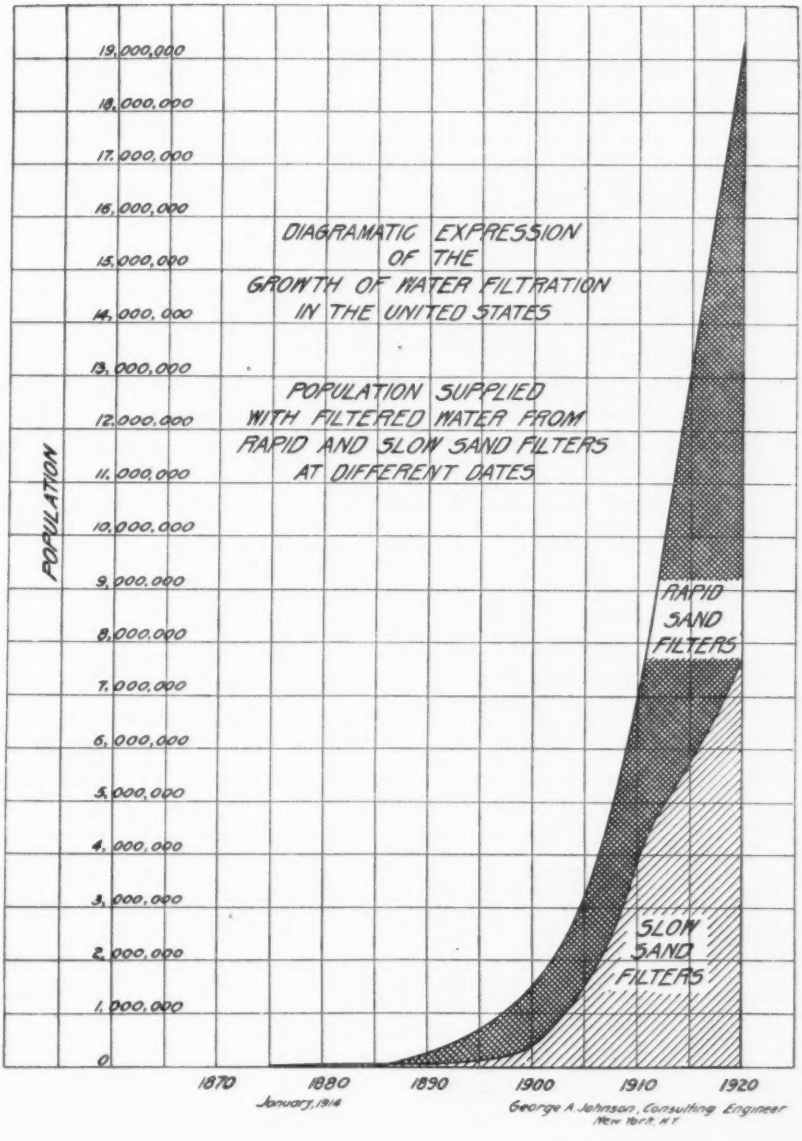
YEAR	TOTAL URBAN POPULATION IN PLACES OF MORE THAN 2,500 INHABITANTS	POPULATION SUPPLIED WITH FILTERED WATER			PER CENT OF URBAN POPULATION SUPPLIED FROM		
		From slow sand filters	From rapid sand filters	Total	Slow sand filters	Rapid sand filters	Total
1870	.....	None	None	0	0.00	0.00	0.00
1880	13,300,000	30,000	None	30,000	0.23	0.00	0.23
1890	21,400,000	35,000	275,000	310,000	1.16	1.28	1.44
1900	29,500,000	360,000	1,500,000	1,860,000	1.22	5.09	6.31
1910	38,350,000	3,883,000	6,922,000	10,805,000	10.13	18.05	28.18
1914	42,500,000	5,398,000	11,893,000	17,291,000	12.70	27.98	40.68
*1920	48,200,000	7,670,000	19,350,000	27,020,000	15.91	40.15	56.06

* All figures on this line estimated, and based on the assumption that rates of increase during the entire period 1910–20 will be in proportion to those during the period 1910–14.

Population of the United States supplied with water purified by slow sand filters and rapid sand filters at different dates

POPULATION SUPPLIED WITH FILTERED WATER				INCREASE SINCE LAST DATE			
Year	From slow sand filters	From rapid sand filters	Total	Slow sand filters	Rapid sand filters	Total	Per cent which rapid sand filters increase was of the total increase
1880	30,000	0	30,000	.....	.....	.....	.....
1890	35,000	275,000	310,000	5,000	275,000	280,000	98.2
1900	360,000	1,500,000	1,860,000	325,000	1,225,000	1,550,000	79.0
1910	3,883,000	6,922,000	10,805,000	3,523,000	5,422,000	8,945,000	60.6
1914	5,398,000	11,893,000	17,291,000	1,515,000	4,971,000	6,486,000	76.6
*1920	7,670,000	19,350,000	27,020,000	2,272,000	7,457,000	9,729,000	76.6

*All figures on this line estimated, and based on the assumption that rates of increase during the entire period 1910–20 will be in proportion to those during the period 1910–14.


DIAGRAMATIC EXPRESSION
OF THE
GROWTH OF WATER FILTRATION
IN THE UNITED STATES
POPULATION SUPPLIED
WITH FILTERED WATER FROM
RAPID AND SLOW SAND FILTERS
AT DIFFERENT DATES
POPULATION
19,000,000
18,000,000
17,000,000
16,000,000
15,000,000
14,000,000
13,000,000
12,000,000
11,000,000
10,000,000
9,000,000
8,000,000
7,000,000
6,000,000
5,000,000
4,000,000
3,000,000
2,000,000
1,000,000
0
RAPID
SAND
FILTERS
SLOW
SAND
FILTERS
1870
1880
1890
1900
1910
1920
January, 1914
George A. Johnson, Consulting Engineer
New York, N.Y.

DISTRIBUTION OF POPULATION SUPPLIED WITH WATER PURIFIED BY RAPID AND SLOW SAND FILTERS, AND DISTRIBUTION OF PLANTS OF THESE TYPES

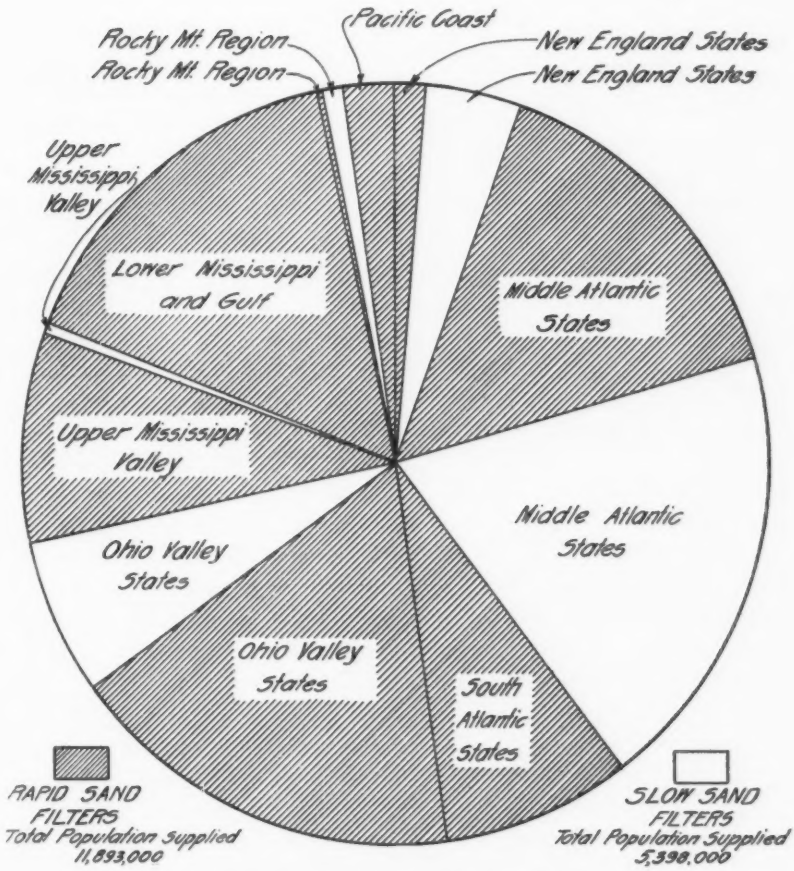
The next two diagrams will serve to show the manner in which the population supplied with filtered water is distributed. In New England, more nearly the rightful home of slow sand filtration than any other district in America, the population supplied by slow sand filters is greater than that supplied by rapid sand filters, while the total capacity of the latter plants is somewhat larger than that of the former. In the Middle Atlantic States the rapid sand filter is superior to the slow sand filter in point of population supplied and capacity of plants. In the South Atlantic States there are numerous rapid sand filter plants, but none of the slow sand type. In the Ohio Valley the rapid sand filter predominates, Pittsburgh being the largest representative of the slow sand process in that district. The upper Mississippi Valley is represented almost exclusively by rapid sand filters, and the Lower Mississippi and Gulf Region is entirely in the rapid sand filter class. In the Pacific Coast and Rocky Mountain Region the rapid sand filter predominates.

SUMMARY AND CONCLUSIONS

To the writer the data at hand furnish indubitable evidence of the obsolescence of slow sand filtration, and the steady rise of rapid sand filtration. When taking for comparison the relative growth of filtration in the United States by the two methods, it is clear that from the beginning the rapid sand method has predominated until, at the present day nearly 69 per cent of the total population of this country supplied with filtered water are served by rapid sand filters.

The first municipal slow sand filter in this country was built in 1875, and the first rapid sand filter in 1885, Since those dates the average annual number of people served by slow sand and rapid sand filter plants was 145,000 and 425,000, respectively, showing in this comparison the rapid sand filter practice to have been the superior of slow sand practice by some 200 per cent.

The decade 1900–1910 was a remarkable one in the line of water filtration development. Slow sand filters were built in many large cities, including Pittsburgh and Philadelphia, Pennsylvania, and Washington, District of Columbia; and rapid sand filter plants were placed in operation in such cities as Cincinnati, Columbus and

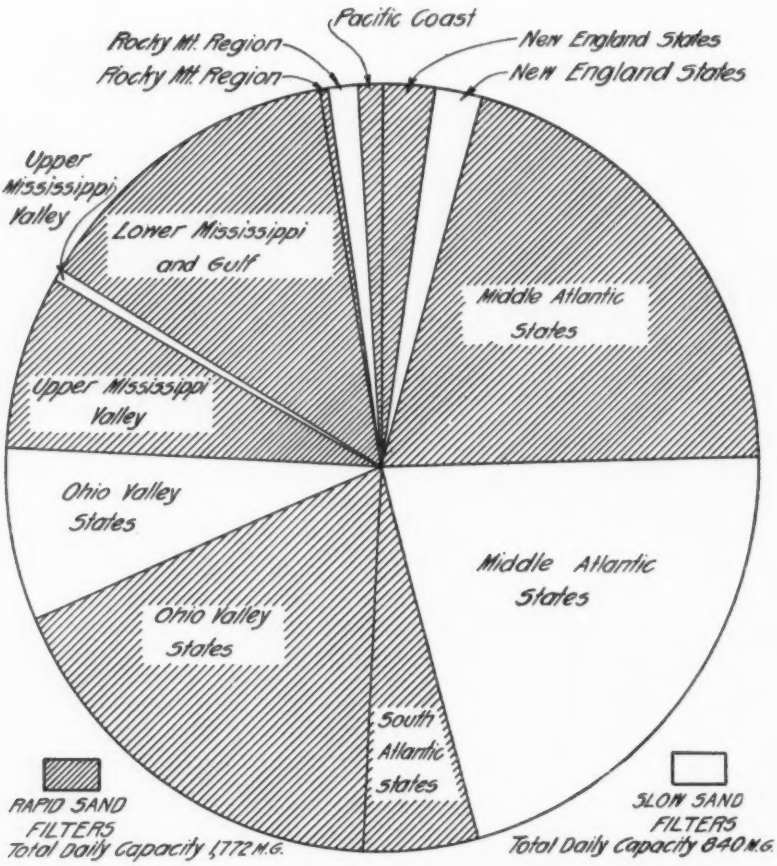


GEOGRAPHIC DISTRIBUTION
OF POPULATION SUPPLIED WITH WATER
PURIFIED BY
RAPID AND SLOW SAND FILTERS

RELATIVE POPULATION
SUPPLIED BY DISTRICTS

January, 1914

George A. Johnson
Consulting Engineer
New York



GEOGRAPHIC DISTRIBUTION OF FILTRATION PLANTS OF RAPID AND SLOW SAND FILTER TYPES

RELATIVE DAILY FILTERING CAPACITIES BY DISTRICTS

January, 1914

George A. Johnson
Consulting Engineer
New York

Toledo, Ohio; Harrisburg, Pennsylvania; Louisville, Kentucky; New Orleans, Louisiana; and Paterson, New Jersey. All told, in that decade filter plants were built to serve a total of 8,945,000 people. When it is recalled that at the beginning of that decade only 1,860,000 people were supplied with filtered water, the contrast becomes particularly marked.

Since 1910 progress has continued. Filter plants have been built and placed in operation, or are now under construction in such cities as Baltimore, Maryland; Dallas, Texas; Evanston, Illinois; Grand Rapids, Michigan; Minneapolis, Minnesota; Portsmouth, Ohio; St. Louis, Missouri; Trenton New Jersey; and elsewhere. All of these plants are of the rapid sand type, and it is significant that no new slow sand filter plants of size have been built since 1910, the work on filter plants of this type having been for the most part restricted to alterations and improvements whereby the efficiency of existing plants would be improved through the addition of coagulating or other preparatory processes.

From these facts it is readily seen that the rapid sand filter process not only predominates at this date, but bids fair to supplant before long, in a number of instances, the existing slow sand filter plants.

It has been demonstrated by actual experience that slow sand filters without the aid of costly preparatory treatment, cannot efficiently purify such waters as those of the Hudson River at Albany, the Delaware at Philadelphia, the Potomac at Washington and the Allegheny at Pittsburgh. And even with preliminary filters there are cases where not only the appearance of the slow sand filtered water is at times unsatisfactory, but high bacterial efficiency at such times is obtained only with the aid of a supplementary sterilization process.

The hygienic efficiency of water filtration processes, measured by the reduction in typhoid fever, may be said to be about 70 per cent. There is little to choose between the average well built and operated slow sand and rapid sand filter, although such advantage as there is rests with the rapid sand filter process which *demands* expert supervision, as should all means operated to the end of saving human life. Slow sand filters, even the simplest kind, and particularly those modified after the fashion of Philadelphia, Albany, Washington and Pittsburgh, *also demand* expert supervision. It is folly to consider the matter in any other light.

Questions of cost alone should never govern the manner of dealing with problems affecting public health and comfort. It so happens,

however, that slow sand and rapid sand filtration cost about the same, with the advantage of lower cost usually in favor of the rapid sand process. A human life is customarily valued at $5,000, and a difference of even $15 per million gallons for water purification, one way or the other, amounts in a year's time to about the value of one human life.

Years of personal study, and reasonably thorough familiarity with the performances of both types of water filters in scores of cities, forces the writer to the conclusion that in the efficient and economical solution of the vast majority of water purification problems in the United States, the rapid sand filtration process is superior to the slow sand process. In view of the general trend during the past ten years or more toward the rapid sand process, and away from the slow sand process, it would seem that the writer is by no means alone in the conviction which he holds.

King James, it will be remembered, used to call for his old shoes, for they were easiest for his feet, and there are some who still prefer the old type filter for the reason that they believe it will run itself, and is therefore less of a constant bother; but in view of the experience of the last twelve or more years the writer prefers the newer process of water purification because there is *no question* about its requiring constant skilled attention; because it is sure in its action; because it insures a minimum typhoid fever death rate coincidentally with the production of a clear, colorless and palatable water; and because it is no more costly than the older type of filtration process.

When one buys an expensive piece of machinery he doesn't turn it over to the questionable mercies of an unskilled mechanic. It is the same with a filter plant which costs far more money. The argument favoring the slow sand filter because it *cannot* go wrong is fallaceous and nonsensical. If properly operated it also requires skilled supervision; and most of the so-called slow sand filter plants of today require a good deal of that.

REMARKS ON THE THEORY OF THE PITOT TUBE

By N. W. Akimoff

Despite the great number of valuable papers and results of tests published during the past few years on the subject of that most useful instrument, the Pitot tube, the hydraulic engineering fraternity has not as yet come to any definite agreement as to whether $h = \frac{v^2}{2g}$ or $h = \frac{v^2}{g}$ is the correct formula underlying its action.

The writer has been often astonished at the extreme violence, generally characteristic of discussions on this apparently innocent subject. Take the most peaceful and gentle spirited hydraulician, convinced, for instance, that $h = \frac{v^2}{2g}$ is the right formula, and suggest to him a mere possibility of constructing an apparatus that will yield $h = \frac{v^2}{g}$. Will not his gentleness depart instantly? Will he not accuse you of ignorance, of the most malicious juggling with facts, of deliberate twisting of the most sacred opinions of classics?

As a matter of fact anyone can make a rod that will work according to $h = \frac{v^2}{g}$, although not every rod, taken at random and under all conditions, will yield this result. It has therefore occurred to the writer that the bitterness, inherent to the controversies as to whether $\frac{v^2}{g}$ or $\frac{v^2}{2g}$ is correct, is mostly based upon the fact that entirely different premises are at the botton of such discussions.

The formula $h = \frac{v^2}{g}$ is the same as $v = 0.7\sqrt{2gh}$ and it is not very difficult to build rods that will yield this result or even slightly less, instead of $v = c\sqrt{2gh}$, where c varies from 0.84 down to 0.75, as is the case in some of the rods now on the market.

It is often "assumed" that c really ought to be unity and, therefore, that smaller values of c are caused by the "suction," due to the "trailing orifice," bent back (Fig. 5, c) in the direction of the flow.

It so happens, however, that out of all means available for decreasing the value of c, the "trailing orifice" seems to be the least effective; experiments made both in this country and abroad show that the lowest c that can be secured with the trailing orifice is 0.84 and often as high as 0.92, whereas, as said before, other means enable us to bring c down to 0.75 and even to 0.70. Greater "reliability" is claimed by the advocates of the "trailing orifice," however, which point it is not our intention to consider in this paper; our object being to present a few considerations relative to the formula itself of the Pitot tube,

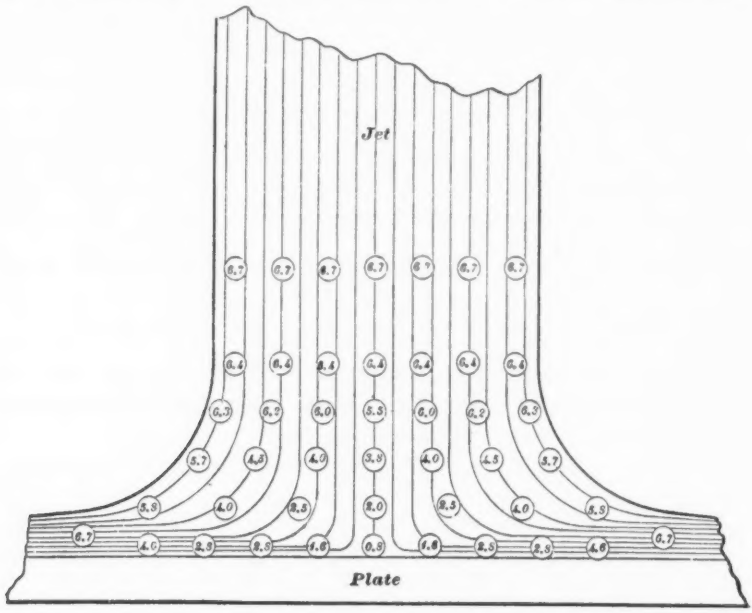


FIG. 1. STREAM LINES AND VELOCITIES OF A JET IMPINGING ON A PLATE

and not at all either to indorse or to condemn any particular existing article now on the market.

We shall first take up the impulse tube itself; the "static" tube will be considered later. The writer will assume that everyone is familiar with the article on "the Pitot Tube; Its Formula" by Mr. W. M. White, published in the *Journal of the Association of Engineering Societies*, August, 1901. This article, which in our time will perhaps be found somewhat unconvincing, and, possibly a trifle obsolete, contains at least one valuable feature, which will be of advantage in

our discussion. In order to substantiate his opinion that in an impact tube, whose impinging surface is one of revolution, the coefficient c, of conversion of velocity head into static head is exactly unity, Mr. White has made many tests, the results of one of which are given in a chart (Fig. 1), taken from his paper.

A stream of water was directed against a round plate; individual velocities of separate filaments were measured and the results, in feet per second, are marked on the chart.

At the present time it would not be necessary to take all this trouble, the effects of the stream, directed both against a long, narrow strip (dam) and against a round plate, have been carefully analysed with the following most interesting results (Fig. 2, for round plate only).[1]

1. The stream lines S are curves of third degree, possessing this most curious property: that all cylinders, inscribed in the surface generated by each stream line are equal, so that, for instance, the volume of A-B is the same as that of C-D. The stream lines begin to diverge at a considerable distance from the plate. Of course the size of the plate proper does not in the least affect the general shape of the curves.

2. The curves of equal velocity EV are ellipses, located as shown and having O as center. Since, in general, these lines intersect the stream lines at two points, such as F and G, it is clear, that somewhere between F and G there must be a point of minimum velocity, where the corresponding ellipse is tangent to the stream line. The locus of such points of minimum velocity will be a straight line, OH, the angle of which with the base will be about 20°.

3. The curves of equal pressure PP will likewise be ellipses, but their common centre will be located below O and is not shown on our drawing. The curve of maximum pressures, (that is of points on each stream line, where the pressure is greatest for *that* curve) will be a hyperbola YOY, tangent to the plate at O.

Careful attention must be paid to the fact that the straight line OH, of minimum velocities, and the hyperbola YOY (of maximum pressures) do not coincide, so that minimum velocity does not mean greatest pressure, which excessive freedom with Bernoulli's theorem might lead us to think would always be the case. We readily forget that Bernoulli's formula has been established for parallel flow only

[1] This construction is given in several books on advanced Hydraulics. It was originated by Professor Prasil and is laborious, although not especially difficult.

and that it does not hold good for any other kind of flow; at least not in the shape in which it is given in books on hydraulics.

Such, then, is the correct aspect of the phenomenon of a jet impinging upon a round plate. A plate, long and narrow, would mean a somewhat different distribution of velocities etc. and all lines would appear somewhat different from those given in Figure 2. It follows, therefore, that contrary to the established assumption, the point O cannot possibly be considered the true impact point. Here the "static" pressure is indeed maximum, but the velocity is zero and therefore it is really the ideal point from which to determine the true static pressure, undisturbed by any "suction" or "trailing" effect.

It will of course be understood that this static pressure represents the conditions somewhere "up stream" from the plate itself; for instance in Mr. White's experiment, it simply shows the height from which the water falls. In other words we have here not the "velocity due to a certain head" but the head itself. This may be made to serve as a check, but in itself is not especially instructive or interesting in our present problem. But then, it is perfectly possible and feasible so to calibrate almost anything of this sort as to give fairly good results; only, our layout, Figure 2., or, for that matter, the "practical" sketch, Figure 1, as given by Mr. White, shows that the middle point is merely subjected to the action of dead water. We might as well have a stationary column at that point.

Any other point, in the vicinity of the curve YOY, of maximum pressures, will be more likely to register the effect of impact, due to velocity, in addition to static pressure at this point.

All the foregoing refers, of course, to a stream acted upon by gravity and directed downward on a horizontal plate, in other words to the condition, illustrated in Mr. White's sketch. In a pipe, with water flowing under pressure, the foregoing conclusions may require suitable corrections.

It is nevertheless quite obvious that the point which is generally assumed to be the true impact point, apparently placed against the greatest action of the stream, may prove to be at a great disadvantage, so far as the determination of velocity is concerned.

The size of the plate, as has been already stated, does not enter into our deductions; and the nozzles B, C and D (Figure 3, taken from Mr. White's paper) will naturally cause the same distribution of velocities and the same effect. The nozzle B is in the same class, because our mere desire to "catch" the energy by means of a funnel

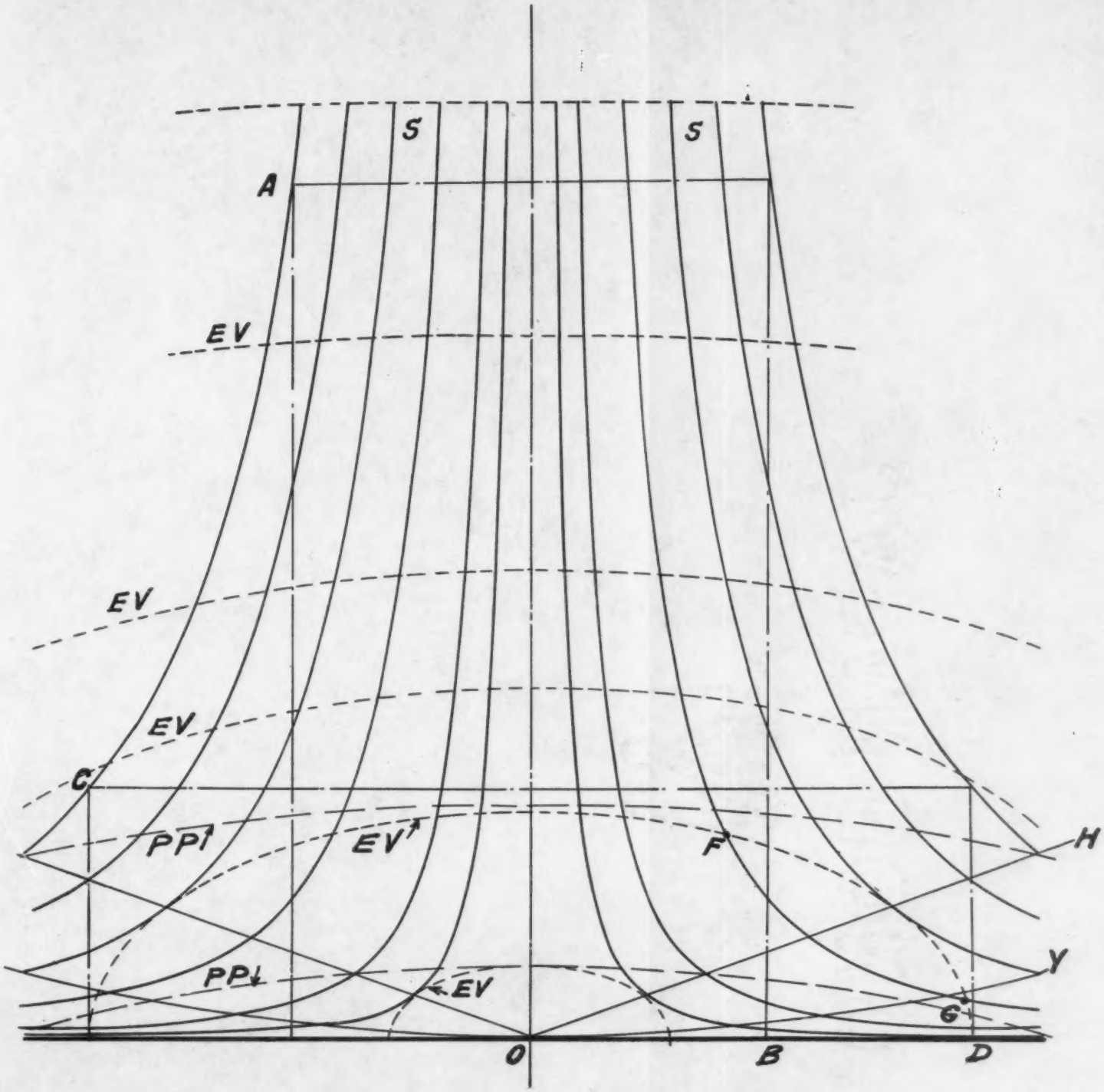


FIG. 2

shaped orifice does not in the least alter the fact of the central stream being perfectly neutral, while other stream lines may, indeed, assume a somewhat more fanciful shape than that given in Figure 2.

So far as the nozzle A is concerned (Darcy's shape), it must be remembered that, unless the nozzle itself is reasonably long, the fittings, etc., back of it should not be neglected. It will be easily seen that a large T or L piece immediately back of a very thin nozzle will cause the formation of stream lines similar to the foregoing, with the same inevitable effect, viz: the neutral stream, possessing the greatest static pressure, but inert, dynamically.

That the rod itself, beside the nozzle proper, will exert a certain influence upon the results, has been very clearly demonstrated by Professors Easby and Pardoe of the University of Pennsylvania. In

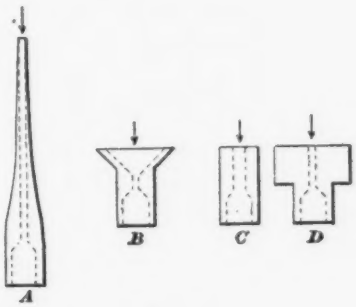


FIG. 3

their experiments a comparatively small pipe was chosen and the rod was of uniform blade-like cross-section. By means of suitable stuffing boxes the rod was made to pass completely through and extend on both sides of the pipe; so that traversing the pipe meant merely shifting the position of the orifice, and not, as in other rods, the immersion of an additional portion of the rod proper. Under these conditions the traverse curve was materially different from the one usually obtained from such small pipe (4 inch). The curve was visibly more symmetrical and thus proved, beyond doubt, the experimentors' point.

The fact has been long established in theory and confirmed by experiments of all kinds, that the *force* (*not* the pressure per square inch) due to an impulse of a jet is $M \times V$, mass times velocity, where the mass is the second-weight divided by 32.2 and the velocity is in

feet per second, so that the result is in *pounds*. This immediately leads to $h = \frac{v^2}{g}$, in other words to the conception of *twice the head generated by the velocity*.

That $M \times V$ is the actual force, due to an impulse, can be clearly proved by an experiment, similar to that described in Professor Merriman's *Hydraulics*, in the chapter on "Dynamic Pressure of Water," where a jet, acting upon a plate in front of an orifice, is capable of sustaining twice the head, to which the jet itself was due (Figure 4).

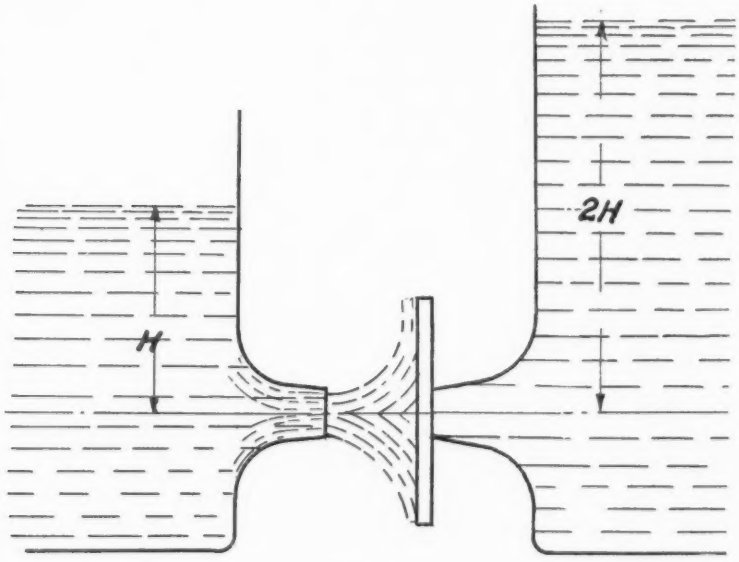


FIG. 4

Should the plate be removed the head in both vessels tends to the same value, as was almost exactly demonstrated by Professor Unwin; but does such a condition correspond to our initial problem?

In an experiment of this kind, can we possibly expect any result other than an equilibrium, subject solely to corrections due to frictional and other losses?

Zeuner, if an impartial, conservative authority is to be quoted, in his well known book on *Practical Hydraulics* (German or French),

gives 1.25 as a safe value to be used in $h = \zeta \frac{v^2}{2g}$ while demonstrating that its theoretical value is $\zeta = 2$, he likewise cites a few tests made both by himself, in connection with turbine work, and by others, in experimenting on water scoops etc, where the value of this coefficient was actually 1.25. Was this because the whole jet was utilized instead of only a small central filament?

Summing up the foregoing, therefore, and without the slightest desire to question the accuracy of the numerous tests, made by well known experts, the writer does not feel that, so far, there is sufficient evidence for the fact that in a single tube $h = \frac{v^2}{2g}$ is the greatest pressure that can be secured; and certain experiments, although made in perfect good faith, rather tend to demonstrate the contrary. So much for the impulse effect.

So far as the suction is concerned, we shall begin by referring to Mr. Ferris' discussion of the same paper by Mr. White. The following are Mr. Ferris' conclusions: (1) The suction effect of the trailing orifice is very little greater than that of the plain static opening, drilled through the pipe wall; and (2) the head, obtained by him was much greater, about 60 per cent, than $\frac{v^2}{2g}$.

These remarks are very much to the point and we shall now endeavor to generalize them. To begin with, analyzing the suction effect due to the trailing orifice arrangement, one cannot help arriving to the conclusion that, owing to the comparatively low velocities, under which the Pitot tube is operated in practice, there is but little hope to secure any appreciable amount of suction by merely bending back the static tube.

Even in theory it can be proved that there is a certain minimum of velocity, beyond which no suction can possibly take place (see Zeuner's book, hereinbefore mentioned).

But there is another point not to be overlooked; the so called "static" opening, which is sometimes drilled in the pipe or else otherwise finished flush with a horizontal plane, cannot help registering *less* pressure than that corresponding to the "static" head, except, of course, for extremely low velocities. Here water is made to shoot past the opening and it is at this point that some suction effect might reasonably well be expected.

By way of illustration, beside Mr. Ferris' experiment, already mentioned, we might cite a few careful experiments made in Charlottenburg (Fig. 5 taken from Dr. Blasius' article, see *Die Turbine*, January 20, 1910). By k is meant the coefficient in $h = k \frac{v^2}{2g}$. It will be observed from c that the arrangement, which from the standpoint of "trailing" effect should yield the greatest k, actually is much inferior in this connection to the arrangement shown on sketch a, where the static pipe is finished at right angles to the flow.

It is perfectly easy further to intensify the real suction effect. It is well known that, as explained in hydrodynamics, a sudden change in the direction of velocity can cause very abrupt changes of

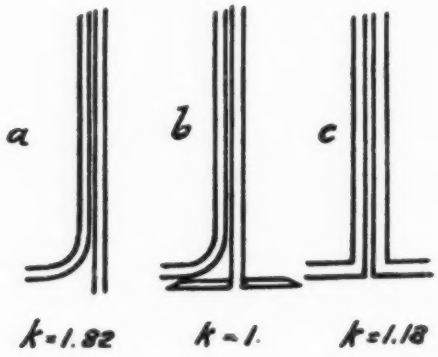


FIG. 5

pressure. For instance, in flowing around a sharp corner, even moderate velocities will cause zero pressures or even negative pressures (vacuum, and corresponding interrupted flow). This, then, seems to the writer to be the correct principle on which to work to secure the real suction effect, so that the value of c can be reduced to 0.70 and lower (in $v = c\sqrt{2gh}$).

This has been experimentally demonstrated by Mr. Paul Lanham, engineer in charge of pitometer division at Washington, D. C., who had a special rod made according to a sketch suggested by the writer. In fact the coefficient given by this rod, seemed to be a trifle less than 0.70, although this rod was not standardized beyond establishing the above point of giving a very low value of c.

It is the writer's hope that the foregoing may be accepted as an explanation of the fact that it is doubly possible to obtain more than $\frac{v^2}{2g}$ in Pitot tube experiments, that is to make each tube contribute its own share toward increasing the pressure difference.

So far as the open channel tests are concerned, where the Pitot tube is submerged only a few inches and is further preceded by butcher knives etc., and is followed by boats or rafts, such tests cannot be of much value except under precisely similar conditions.

Here the observed level of water is always supposed to be the same, so that there cannot be any question of registering any increased pressure as in pipe tests. Here we really trespass upon the domain of one of the most interesting and most difficult branches of hydrodynamics, the *Theory of Waves*, and should abandon all hope of "reasoning" out the problem by means of a mere misuse of Bernoulli's equation.

It is earnestly hoped that the day may not be far distant, when it will be found impossible for an engineer to do without a few elementary notions of hydrodynamics and that there may be a decided demand for well written books on the subject, which are now so sadly lacking in the engineering bibliography of this country.

The writer likewise hopes that some of the latter may contain a chapter or two on the real theory of the Pitot tube, a theory which will give a detailed explanation of the results of careful observations, made in some manner, uniformly agreed upon.

TESTING OF CHECK VALVES

By J. Walter Ackerman

In the year 1908 the water department of the city of Auburn caused an examination to be made of the check valves of the mill and factory connections in the city of Auburn. And as these check valves were of the type that are bought on a competitive basis, and simply represented a design made looking towards an economy of material and labor, the results of this examination can be easily imagined.

It was found in many cases that even the locations of the valves were unknown to the factory, and some had been buried in the ground for a period of twenty years, without ever having been examined. The final result of this inspection was that each and every mill or factory using city water for fire protection, and having a polluted auxiliary supply, was required to install double check valves of a special design, and to be placed in a brick or concrete vault, accessible at all times to inspection and test.

The specifications under which the check valves were made, require that the seat ring, disc holding ring, and screws, and in fact all moving parts, and the hinge pin and bushings of same be of bronze, the disc to be faced with medium hard rubber, and liberal clearance to be made all around the clapper, the pin, the arm, etc. These specifications were largely determined by the inspection department of the Associated Factory Mutual Fire Insurance Companies of Boston, and they also had to meet with the approval of the water department of the city of Auburn.

The regulations are that they shall be tested once every six months by the water department, and taken apart once each year and examined for defects or obstructions, and during this latter inspection a representative from the Associated Factory Mutual Fire Insurance Company is present to observe the condition of the valve and assist at the test.

It is specified by the water department that the inspection and test shall be at the expense of the factory or mill where the tests are made.

Figure 1 shows a diagrammatic representation of an installation made in the city of Auburn. The stop gate valves *A* and *D* are so

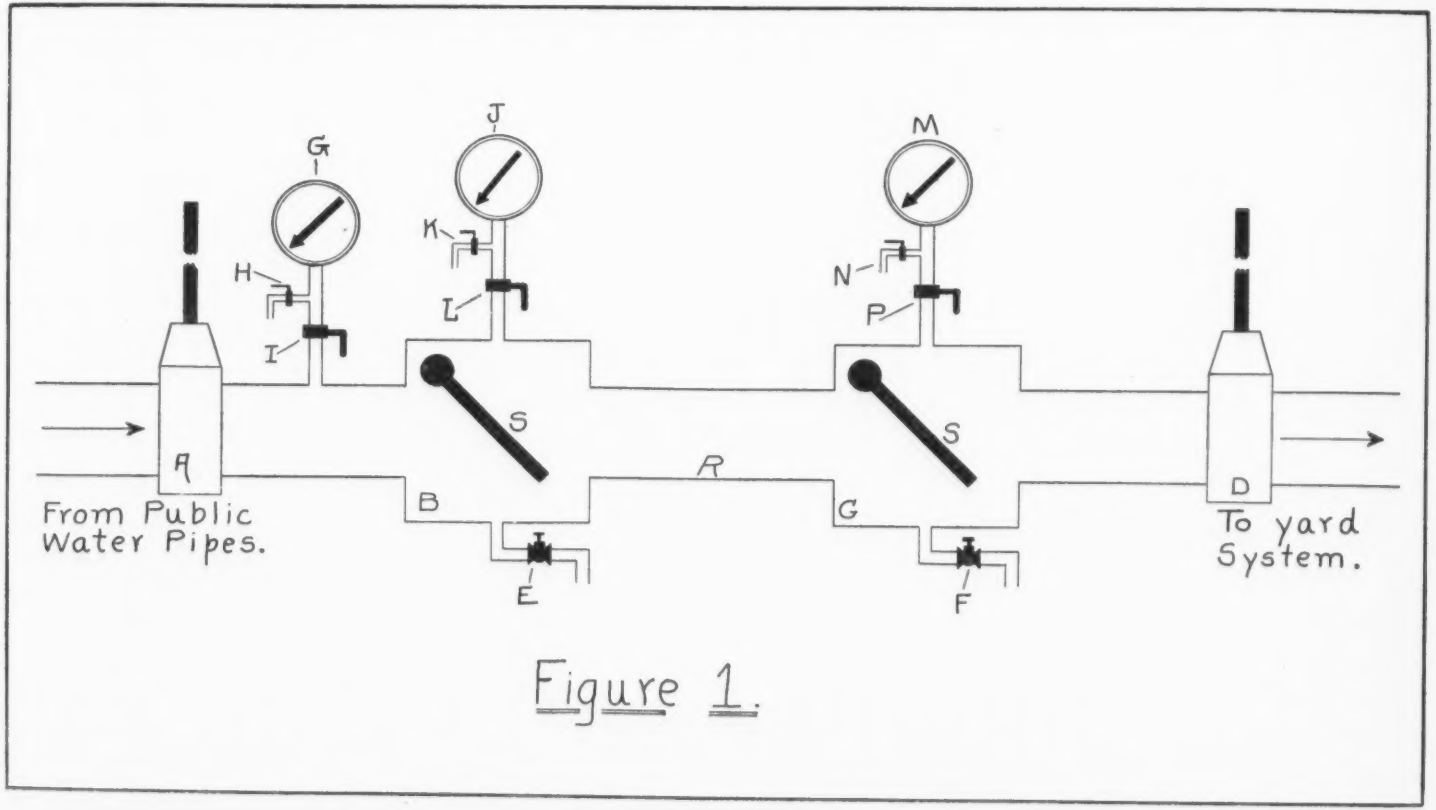

G
J
M
H
K
N
I
L
P
S
S
A
R
D
From Public
Water Pipes.
B
G
To yard
System.
E
F

Figure 1.

placed in line as to absolutely shut off the water from the check valves. On the entering side, between the first valve *A* and the check valve, is a tap with cock and air vent, on which is placed the pressure gauge *G*. Likewise, two other taps on the two check valves are made with the same type of connection. Tapped into the bottom of each of the check valves is a $\frac{3}{4}$ inch connection, controlled by cocks *E* and *F*.

The actual test is made as follows: Closing gate *A*, either the cock *E* or the air cock *H* is opened. The pressure at *G* only should fall if the check valve *B* is tight and the gate *A* is tight. It sometimes happens that the opening of air cock *H* releases the pressure so slowly that the back pressure on check valve *B* will not seat the clapper *S* in valve *B*, and the gauges both *G* and *J* will fall; while if the $\frac{3}{4}$ inch cock *E* is opened the valve *B* will close tight. This simply gives the degree of sensitiveness of the check valve, and indicates its condition; for if the air cock does not show it absolutely tight, the drip *E* may. This generally shows that the rubber facing on the clapper is either coated with slime, or some small indentations occurring, allow the water to pass back through this valve. The conditions of tests are slightly different, depending upon whether a mill has one connection or two. If it has two connections there is always present the city pressure from the other connection coming back to the two valves; whereas, if there is only one connection there is only the static head, or else the fire pumps in the mill are started, and pressure created in that way. If the flow from *E* or *H* is continuous, it is an indication that there is a leak either through both check valves or through the stop gate *A*. If the pressure with *E* and *H* open drops on *G* only, and not on *J* and *M*, it is the gate *A* leaking; but to make absolutely sure, allow all the gauges to assume the normal pressure, and then suddenly release *F*. If gauges *J* and *G* drop, and *M* does not, and flow continues, then it is an indication that the leak is through *A*, and not through *B* and *G*.

After the condition of gate valve *A* and check valve *B* have been thoroughly determined by the above method, then the opening of air cock *K* or drip *F* will disclose whether there is a leak on check valve *G*. And, as before noted, if it shows a leak with *K* open, and tight with *F* open, it simply does not seat properly under a low differential pressure. And this is important, because the valve is called upon to act when the fire pump raises the pressure above that of the water in the city mains. And there always will be a time when the pressure

on each side is just equalized, and then the pressure on the mill side will begin to build up above that of the city side. And while the test may show from both the drips a tight condition, it will not show a tight condition when only the air cocks are used slowly, allowing the difference in pressure to be gradual.

In order that a number of trials may be made on check valve *B*, pressure should be taken from some source and introduced between the two valves. It is preferable in this case to take the pressure from the main on the mill side between check valve *G* and the mill or factory and allow it to come in through either the drip pipe *F* or some other place, in order to balance the pressure each side of check valve *G*. Then this by-pass connection would furnish water for unlimited trials on check valve *B*, independent of check valve *G*, for any water which leaks through *B* would be replaced by the by-pass connection, which, if it was not there, would soon be exhausted if check valve *G* was tight.

The actual value of the test is obtained after it is completed and the examination made of the interior to show actual conditions, such as tuberculations on the iron body, the smoothness of seat ring, the condition of the rubber facing on clapper, and the manner of the closing of the valve. And while the test is only to show working conditions, the examination shows the way the valves are living up to the requirements imposed upon them.

The rules in our department which control the installation of check valves and kindred appurtenances, are as follows:

No water pipes in any building or premises supplied with water by the city shall be allowed to receive a supply of water from any other source except in case of pipes used exclusively for fire protection that conform to the requirements given below:

No connection of the city mains shall be allowed with pipes having another source of water used for miscellaneous purposes except to a tank open to the air, nor shall such pipes have any physical connection with the pipes used for fire protection.

Between the pipes used exclusively for fire protection and the city mains there shall be two check valves placed between two positive valves and located in an accessible brick or concrete pit, with such valves, gages and connections as may be prescribed by the water board for each installation and for the purpose of testing. The check valve shall be of a type approved by the water board. The check valves shall be kept in working order by the consumer and shall be at all times subject to inspection and test by the superintendent of water works, the cost of such test being borne by the consumer. The pure water pipes that have direct connection with the city supply, before the same have

passed through the aforesaid double check valves, shall be painted BLUE; the pipes used exclusively for fire protection shall be painted RED; and the pipes carrying the impure water and that used for miscellaneous mill purposes but not for drinking shall be painted YELLOW. These different systems of pipes shall not have any physical connection with each other in the mill or factory.

There shall be placed on the discharge main of the fire pump a check valve and positive gate. The latter shall be kept closed and sealed by the superintendent of water works, and opened only in case of fire or for the purpose of occasionally testing the fire system, but notice of such occasional test shall be given to the superintendent of water works, and during the test the positive gate between the fire system and the City main shall be kept closed. During the weekly trials of the pump prescribed by the fire underwriters the water shall be discharged either through the relief valves of the pump or through hose connections from the pump.

EQUITABLE HYDRANT RENTALS AND BETTER METHODS FOR APPORTIONING FIRE PROTECTION COST

By John W. Alvord

Probably no one thing has contributed so much to the friction between private water companies and municipalities as the common and antiquated method of paying for fire protection at a fixed sum per hydrant. It is the purpose of this paper to point out some of the evils resulting from unscientific apportionment of fire protection costs, and show how they can be remedied.

In the formation of so many new utility commissions as are now created, and are being created, there arises the opportunity to obtain a more equitable and scientific adjustment of rates than has hitherto prevailed. The Wisconsin commission has already done valuable work in showing that many of the municipalities in Wisconsin do not pay enough for public fire protection; that is to say, that the cost of the public fire protection system is in considerable part loaded onto the private consumer. It is a matter of common knowledge among those who study this subject that this is largely true all over the country. This condition doubtless arose from the fact that when franchises were originally granted, it was easier for city councils to grant and water company promoters to obtain concessions for a slightly larger domestic rate, which came from many small consumers, than a proper and just public payment coming from a usually insufficient and ordinarily depleted general fund.

The injustice of this proceeding is fully evident when a careful study of the two services is made. The valuable paper on this subject by Messrs. Kuichling, Hawley and Metcalf, presented to the American Water Works Association at the Rochester meeting, shows generally that with the smaller cities fire protection cost rises to a 65 or 75 per cent proportion of all the service, while in the larger cities this may be as low as to 20 and 25 per cent.

The Wisconsin commission has, as the result of its studies, generally raised the fire protection cost, and correspondingly lowered the domestic rate in the following cases:

DATE OF REPORT	CITY	POPULATION 1910	PER CENT OF WATER WORKS CHARGED TO FIRE SERVICE	ANNUAL CHARGE FOR FIRE SERVICE
1910	Jefferson, Wis.	2,582	75.0	$2,560
1910	Ripon, Wis.	3,739	65.0	6,082
1911	Oconto, Wis.	5,629	57.5	9,812
1909	Ashland, Wis.	11,594	54.5	20,480
1911	Janesville, Wis.	13,894	54.0	15,795
1911	Beloit, Wis.	15,125	48.0	14,863
1910	Madison, Wis.	25,531	49.6	About 17,000[1]

It is safe to say that in most instances where private utilities are serving the community, the city is paying less than half of the amount which properly should be paid for this service, thus loading the balance of the cost usually on to the small private consumer.

In the case of a municipality which owns its own plant, and has largely paid for it from issues of bonds, paid from annual taxes, the fixed charges are perhaps more equitably distributed, but it is yet an open question, even in such cases, if a careful study would not disclose that the consumer is not paying more than his proportion for the public service, by reason of the fact that he virtually bears all the operating cost.

The proper method of dividing the cost of public and private services is, (1) to make a valuation of the plant; (2) to make a study of the value of a theoretical plant, sufficient in capacity for fire service only, and (3) a study of the value of a theoretical plant, capable of domestic service only. The sum of the (2) and (3) study will give a value much greater than the value of the combined plant, and the value of the combined plant should then be divided between fire service and domestic service in proportion to the theoretical values found necessary in each case. To illustrate: if the cost of a combined fire and domestic service in a single plant is found to be $100,000 for a given community, and the cost of a strictly fire service plant is found to be $80,000, and a strictly domestic supply plant $70,000, or a total for two separate plants of $150,000 for the same community, then the value of the combined plant should be divided between the two services on the basis of eight-fifteenths and seven-fifteenths of the total value of the combined plant, or about $53,400 for the proportionate investment for fire service, and about $46,600 for the proportionate investment in domestic service.

[1] Special local conditions favor a low fire service charge at Madison.

In addition to this study, it is necessary to inquire, in the same manner, into the operating expenses, and divide the combined operating expenses between those chargeable to fire service and domestic service. Here it will be usually found that domestic service carries the higher charge.

Having found the total investment in each service, and computed its interest and depreciation, and added to this the proper proportion of operating expense, we have a proper and equitable division of annual revenue which should be derived as between the private consumer, on the one hand, and the public fire protection expense, on the other, and, as has been before said, it will be usually found that the public is not paying nearly enough for the public service and the private consumer is bearing the burden.

Here, therefore, is an opportunity for our public service commissions to do some scientific rate making, easing the burden particularly on the small consumer and placing it where it justly and properly belongs, on the general public.

But more than this can be done. When the proper amount for the public to pay for general fire protection is properly ascertained, it can be more scientifically applied than under the present irrational method so that it will not result in the constant friction that the present method involves.

The method of paying for fire protection at so much per hydrant is very common, but it is illogical and improper for the following reasons:

1. It does not apply the payments to what the public really pays for, viz., pumping capacity, distribution capacity, and the size and number of fixture openings.

2. The present method has a tendency to contract the fire-fighting efficiency, by concentrating the payment on an outlet fixture, thus constantly tempting the cities to improper economy by the neglect to install the proper number and closer spacing of the hydrants, with increasing fire risk in any given district.

3. The present method spreads the cost of fire protection inequitably, in that Cities have to pay for outlying districts, with low fire protection risks, as much per hydrant as in central business property, with high protection risks.

4. It discourages the extension of the distribution system into outlying districts, by reason of the relatively high cost of the hydrants needed.

5. It does not encourage the use of adequate sized mains, because attention is concentrated on the outlet fixture and its spacing rather than the size and capacity of the distribution system.

6. It discourages the setting of additional and intermediate hydrants when the adjacent fire risks increase, even though capacity of mains may be available.

7. It makes for constant contention between the city and the company whenever new hydrants or new extensions are needed, by reason of the above facts, which neither the company or the city officials are always able to fully analyze or understand. Thus we have these two important interests in constant antagonism where both should be in harmony in working for a common end at a just cost.

Many of the bitterest contests between private companies and municipalities arise from this cause.

Now, all this source of contention and friction can be entirely avoided by a more proper method of payment for the public service, which will eliminate the objections above made, and place the burden of cost where it belongs; that is to say, in the capacity of the system for fire protection rather than on the number and spacing of outlet fixtures. Just as it is more equitable to sell water to the private consumer per unit of volume, by meter, rather than by number of outlets in the consumer's home, so is it more equitable to pay for fire protection per unit of capacity for fire protection purposes rather than mainly by unit of outlet fixture. In other words, let us pay the company for the amount of capacity it has in its pumping and distribution system, as well as for the number of outlets and we will have destroyed most of the sources of friction that now exist.

The method that first suggested itself to the writer was to apportion fire protection cost in part to pumping capacity, in part to pipe distribution capacity, and in part to outlet, but on reflection it was seen that this triple division produced some complexity, which was not really needed, and which did not make for simplicity.

It is comparatively easy for cities to enforce the installation of adequate pumping capacity, and where pumping capacity does not amply exist it is not difficult for water companies to enlarge it in fairly reasonable length of time. Attention is constantly directed to deficient pumping capacity and its future lack is not difficult to foresee.

But the enlargement of distribution pipe capacity up to the level of a constantly growing population is not so easily accomplished. Its

lack is produced by insidious neglect, of which the manager, the administration, and the public has no warning note. It is here, if anywhere, that we need in the ordinary plant and city to have an incentive to upkeep and enlargement. It would, therefore, appear that for purposes of simplicity, it would be fully as effective to proportion fire protection payments in large part on pipe distribution capacity, and in small part on outlet fixture payments, as to proceed in a more complex manner by including pumping capacity.

With this principle in view, and having arrived at the proper amount to be paid the utility for fire protection by the methods herein first described, it remains to apportion this amount in such a way that incentive is introduced to properly install mains and distribution system of the proper size, and hydrants of the proper spacing, so that the fire risks will be proportionately met in different sections of the town, and at the same time, so that extensions and additional hydrant rental will not be burdensome, and, in short, that all or most of the causes of friction between the municipality and the utility enumerated above will be removed.

This can be done by paying for the fire service in large part per mile of pipe at rates depending on its capacity.

For the purpose of having some regulating limit, it is also desirable to pay for the hydrants a small sum, which represents a reasonable return on their cost of installation, together with an allowance for their depreciation and annual maintenance. This in reality amounts to a nominal sum, generally not more than $5 to $8 per annum per hydrant. At this rate it should be the privilege of the city to order in as many or as few hydrants as proper attention to the relative fire risks of the different sections of the city would seem to demand. A proper appreciation of the rapidity with which long lines of fire hose reduces pressure would undoubtedly make for a liberal allowance of hydrants under such a scale of payment.

The company, being paid an annual sum for the hydrants and their maintenance, has no separation of interest from the viewpoint of the municipality.

The bulk of the payment for fire protection under such a system should come from the pipe capacity unit.

The pipe capacity unit does not need to be scientifically exact; it is sufficient for all practical purposes if it is only approximate, and particularly desirable that it be simple.

The best unit which has occurred to the writer, that approximates

in some degree the relative cost and capacity of mains sufficiently for this purpose, and is at the same time simple, is the inch foot of diameter; that is, the number of inches of diameter 1 foot long in a given system of distribution pipe. Thus, a 6 inch pipe is rated at 6 inch feet to every foot, and a 12-inch pipe 12 inch feet to every foot.

Having found the total amount of money to be paid for fire service in a given system, we should deduct the amount that will be raised by the nominal hydrant rental before described, and we should also deduct the amount which will be raised from sprinkler systems and other especial methods of fire protection, and the remainder will be the amount to be paid by the city on the pipe capacity unit or inch foot basis. The total number of inch feet in a given system, divided into the amount to be raised by this part of the protection, will give us the cost per inch foot. This will probably fall in most normal cases within $\frac{4}{10}$ to $\frac{7}{10}$ of one cent per inch foot.

The total annual payment then for any given pipe will be its diameter times the inch foot unit times its length.

Thus, if the inch foot unit is a half a cent, a 6-inch pipe will have a fire protection rental of 3 cents per foot per annum, and a 12 inch pipe will have a rental value similarly of 6 cents a foot annually.

To this amount computed for all the pipe lines should be added the nominal hydrant rental computed on the total number of hydrants, the total making the annual public fire protection bill to the city.

It is true that the cost and capacity of mains increase at a rate greater than the first power of the diameter. The capacity increases as the square of the diameter, and the cost between the first power and the square. If the cost of mains is increased by the replacement cost of cutting through pavements, the adjustment of cost as between different diameters will be closer.

A uniform rate per inch of diameter is a slight inducement toward the smaller pipe, as that has the larger return, and this may be equalized (if it is thought wise) by a sliding scale, but, as a matter of practical effect, it is not believed that the variation will have any appreciable influence in keeping down the capacity of the distribution system.

The advantageous features of this arrangement are:

1. That the city proportions its payment to the amount of distribution pipe capacity it receives, as well as the number of fixture openings (hydrants) that are available

2. The payments for fire protection are usually and normally in

proportion to the fire risks in the different districts, because the larger mains are in a general way either within, or lead to, those districts most fully built up and most needing fire protection.

3. There is every incentive to the company in extending its distribution system, to keep sizes of mains up to the fire requirements, and to reinforce weak districts, and the cities' interest in fire protection and the companies' interest in proper remuneration are thus coördinate.

4. In outlying districts the city does not have to pay for protection out of all proportion to the capacity needed, and it may further regulate the number of hydrants in such districts to suit their character and growth.

5. There is no obstacle to the introduction of new hydrants when given districts need increased fire protection, and thus the fire department is rendered much more efficient.

The writer, in company with Dean F. E. Turneaure of Madison, Wisconsin and Dean Marston of Ames, Iowa, was appointed in 1912 as a board of arbitration to determine the value of the water works property, at Freeport, Illinois, decide upon the necessary enlargement of the plant, fix a schedule of fair rates, and draw up an ordinance for the extension of the franchise, should the city determine to extend rather than purchase.

This opportunity was taken to introduce the method of paying for fire protection described herein, and the city electing to extend the franchise, the method of paying for fire protection has now been in force there for the last two years, and has given satisfaction.

The value of the Freeport plant in April, 1912 was found to be $316,000 (population about 19,000). New extensions and enlargements were recommended which would increase this amount to $502,927. A fair return on this amount was recommended to be such that it would at no time fall below 7 per cent, after due allowance for all expense of operation and depreciation. This called for a return within four years of at least $70,000.

From a careful analysis, it was found in this particular case that 44 per cent of the total investment was necessary for fire protection, and 56 per cent for domestic consumption, and it was recommended that the city raise its payment for fire protection from a total of $10,000 (the amount then paid) to $15,000 in 1915, and $19,800 later. A corresponding reduction was made in the rates to small consumers.

The apportionment of the fire protection cost was made on the basis herein described, and resulted as follows:

For each hydrant the annual sum of $7.50 to cover interest, maintenance, and depreciation.

For future extensions as follows:

$0.10 annually per foot of 20 inch pipe
.08 annually per foot of 16 inch pipe
.06 annually per foot of 12 inch pipe
.05 annually per foot of 10 inch pipe
.04 annually per foot of 8 inch pipe
.03 annually per foot of 6 inch pipe
.02 annually per foot of 4 inch pipe

No pipe smaller than 4 inch was recommended, and 4 inch not to be used for greater lengths than 1000 feet between cross mains.

For private fire protection services:

Private hydrants $7.50 annually per hydrant.
$0.06 annually per foot of 6 inch supply pipe.
.04 annually per foot of 4 inch supply pipe.

The charges per foot of private fire protection supply pipe being higher than charged the city, because such supply pipe affords no revenue from domestic consumers.

For sprinkler service:

An initial payment of the entire cost of the supply pipe, safety valves, etc.

For each sprinkler head $0.05 annually.

Minimum charge $50 per annum.

The above findings are only given to illustrate the working out of the system. Each case requires to be investigated by itself, and may result in very different figures owing to special local causes.

It is to be hoped that the suggestions here contained will commend themselves to those cities or companies about to renew fire protection contracts, so that past causes for fruitless difficulties and friction may be avoided, and especially is it possible for the new public utility commissions to introduce such a method as is here outlined, because they have it easily within their power to adjust rates to a just and scientific basis, equitable to both the public and the private consumer, without bargaining or pressure from political or personal interest.

USE AND BENEFITS OF PRESSURE RECORDING GAGES

By J. M. Diven

Recording gages for keeping a permanent record of steam and water pressures and vacuum have been used by water works for many years. In 1891 Mr. Charles A. Hague, in his introduction of his valuable paper entitled "Value of Pressure Records in Connection with Water Works," said that we should know what is going on inside of steam and water pipes, just the same as we have found out what is going on inside the steam cylinder by the use of the indicator.

In his paper Mr. Hague quotes Mr. Edwin Darling in his 1889 report of the Pawtucket, Rhode Island, Water Works, as follows: "No well conducted water works can afford to be without recording gauges, and, when properly located, they will, in my opinion, pay for themselves within one year" (*Proceedings Am. W. W. A.* Philadelphia, 1891, p. 77).

These extracts go to show that the value and usefulness of recording gages were fully recognized, more than twenty years ago. We certainly do need to know what is going on inside of the steam and water pipes under our charge, and at all times. A look at the non-recording gage shows us the pressure at that time, but is no proof or indication of what it was a minute before. The recording gage does give this information; it tells what the pressure was at any time, day or night, and on any date. It is a constant watchman, just as a water meter is a constant inspector. It keeps a complete and indisputable record of pressure at all times.

IN THE PUMPING STATION

A recording gage on the steam line is the best possible check on the work of the fireman and engineer. If pumping to a reservoir, under a constant load, the uniformity of the pressure line tells how uniformly the fires are being tended and the feed water applied. An absolutely even line would indicate perfection in the fire room, a ragged line, inattention on the part of the fireman. With properly kept boiler room records, the fireman's work can be accurately checked.

A sudden drop in the pressure line should correspond exactly with the record of fires cleaned or boilers blown off. A quick drop or sudden rise at any other time indicates improper firing.

With a direct pumping system, or where pressure has to be raised for fires, a less even steam line is to be expected; but a comparison of the steam and water charts will show how quickly the boiler responds to a sudden demand for more water pressure, by restoring and holding the pressure when the work of the pumping engine is suddenly increased. A direct pressure pumping system requires greater care and alertness on the part of the engineer and fireman, and the recording gages are the best possible means of knowing how well they perform their work. Perfection would mean a line equal to one drawn with a compass or straight edge, according to the style of chart used, and the nearest approach to this indicates the best work. A ragged, zigzag line means inattention and carelessness, or something wrong with the boilers, steam lines, engines or pumps. It rests with the chief engineer or superintendent to find out which is at fault, and to correct the fault; the gages cannot do this, they can only tell in positive language that something is wrong.

The presence of recording gages is an incentive and stimulus to the men to do better work. They know that they are constantly watched; that a constant and indisputable record is being made of their work. The superintendent, when he visits the pumping station, or when the gage chart is placed on his desk, has a perfect check on the work of the pumping plant force, and a record he can preserve for all time. When he has occasion to call an engineer or fireman to account for laxness, he has before him, to confront the culprit with, an undeniable record of his work. A poor pressure line may not be the fault of the engineer or fireman; it may be due to poor fuel, a leaky or sagging steam line, bad feed water, poor draft, or engine trouble; but the engineer is just as much at fault, for not promptly recording and reporting such troubles, as he would be for neglecting his work, or not keeping the fireman up to his part of it.

If good pressure charts are obtained for a while, but constantly and uniformly get bad, it indicates either that the plant is wearing out or being overloaded. If both good steam and water charts are obtained when the plant is pumping say 5,000,000 gallons a day, and grow more and more ragged as the pumpage is increased, they surely indicate that the pumps or boilers, or both, are being worked beyond their economic capacity. If the pressure lines grow in irregularity,

when there is no increase in the work, they indicate that some part of the plant is wearing out or is in bad order, and needs overhauling.

Steady lines of the steam charts indicate uniform and careful firing, and a steam plant in good condition. Ragged gage lines indicate either poor and irregular firing or bad condition of the boilers. If the feed water is a scale forming water, the poor gage lines bear out the evidence of the coal scales, that the boilers need cleaning.

A fireman cannot fill his firebox, then light his pipe and go out of the fire room to seek a cool place, and maintain an even line on his steam gage charts; but must stay by his fires, fire frequently and lightly, and keep clean fires. Knowing how completely he is on record he will be careful, and will also keep a record of everything that might affect his record, such as poor fuel, time of cleaning fires, blowing off boilers, etc. The latter records, which might otherwise be neglected, are in themselves of sufficient value to more than pay the cost of the recording gages, if no other benefits were obtained.

The vacuum gages should also be recording, to indicate the condition of the engines and pumps, a falling off in the height of the vacuum line indicating leaks that mean more coal. The lines on the vacuum gage charts should not vary much. If a vacuum of 27 can be obtained one month or year, it should be maintained the next month and year. If a higher vacuum is maintained with one kind of packing than with others, the one holding the high vacuum is the best, and probably the cheapest, though much higher priced than the others. The difference in cost might be made up many times in the saving of fuel.

No engine room is complete without at least three recording gages, as the operation of the plant cannot be properly checked and governed without them. They also insure better work on the part of the operatives.

Many times the recording gages create a spirit of rivalry between men on different shifts, through an effort on the part of each shift to do better than the others, to show more even lines on the charts; thus causing all to do better and more economical work.

There should be a certain economical relation between the steam pressure and the water pressure. When this is worked out it should be as closely maintained as possible. An unnecessarily high steam pressure, even though it gives a very uniform gage line, is not economical.

ON THE DISTRIBUTION SYSTEM

On gravity supplies or pumping plants, in cities of considerable size, a gage, or gages, should be placed on the distribution mains at some central point, or at typical points. If more than one service, high and low service, each should have a recording gage. It is well, when practicable, to take gage services off of large or leading mains, where they will not be liable to the great fluctuations in pressure frequent on smaller distribution pipes.

Of course there should be a gage in the office, where it will be under the eye of the superintendent when in the office, and, at other times, of some other officer or employe of the water works, of the night watchman, if one is employed; one in the superintendent's house, for reference at night, is convenient, but, with telephone connections, not so necessary.

The connection to the gage should be entirely independent of all other service to the building in which it is located. It should be of good size, not less than $\frac{3}{4}$ inch, and placed where it will be in no danger of freezing, as there cannot be a circulation in the gage service and an accurate record of pressure on the main at the same time, without the expense of a double service, in which a free circulation would be doubtful, unless it were very carefully planned, and even then there would be an item of friction in the smaller pipe, that would be difficult to account for. Of course water cannot be allowed to run in the gage supply pipe, in cold weather, without totally destroying the value of the gage record.

The gage service should be so arranged as to be easily blown out or flushed, to avoid all possibility of stoppage, by an accumulation of sediment. While a $\frac{3}{4}$ inch or larger service is recommended, the gage operates with a very small opening; the object of the large service is to avoid stoppage.

LOCATION OF GAGES

The pumping station gage comes first, and, in smaller plants, is all that is required. Larger plants should also have an office recording gage. Cities or towns with two or more services should have a record on each service. Large cities should have recording gages at various points on the distribution system, notably near the congested value districts.

Gages can be placed in the residences of employes of the works.

They require little attention, winding of clock, changing of charts, and seeing that the pen, where ink records are made, is kept properly filled.

The fire department stations are convenient places for recording gages, and the fireman's interest in the water pressure is enough to insure proper attention. For municipal plants, police stations can be utilized, if they are better located than the fire engine houses, that is, on larger supply mains.

Very long services should, naturally, be avoided, both on account of the expense and the danger of stoppage in long lines of small pipes. The service should be of durable material, and, with corrosive waters, iron services should be avoided, as an accumulation of iron rust would stop up the minute orifice of the gage.

THE BENEFITS OF RECORDING GAGES

The benefits of recording gages are many; they have been known to stop law suits, where actions for damages caused by low pressure were threatened. The recording gage settles many disputes concerning pressure at time of fires or other times. Often complaints of low pressure are made, not enough to supply the upper floors of some buildings. A nearby recording gage demonstrates that it is a local trouble, within the building. These complaints are frequently that the pressure is low at certain hours of the day or night, but a gage chart showing a uniform pressure at those hours, demonstrates that it is local use of water, in the building itself, or in the distribution system near it, that causes the trouble.

The recording gage charts show the effect of cold weather on the water supply, the low pressure lines on the chart in freezing weather indicate the extent to which water is being run to prevent freezing. They also reflect the hot weather use of water; a low pressure line on the chart at night tells of water allowed to run on lawns all night.

PORTABLE RECORDING GAGES

All parts of a city cannot at all times be covered by recording gages, so that the story they have to tell is incomplete. Portable recording gages set up in various places for short periods make a fairly complete record of the conditions of the supply at points on the distribution not covered by regularly established recording gages. Such records in large buildings, factories, etc., where the fire hazard is great, are particularly useful.

A gage set up for a single day gives a valuable record, as it shows a comparison between the day and night pressure at some particular point.

Portable gages can, with advantage, be connected to fire services, as, on such services, they would be free from the fluctuations due to the use of water on the premises, or, if the fluctuations existed, they would reveal improper use of water from the fire services. With a small portable house or box to protect them, gages can, in warm weather, be attached to fire hydrants, to make a record of the pressures on the mains at any point. Summer records of this kind, made on the outlying mains, mains in sparsely built up parts of the city, on long runs of small distribution pipes, and on the outskirts of the town, are of great value and interest.

PRESERVATION OF GAGE CHARTS

Charts should be permanently kept as a part of the records of the works; as part of the history of the plant. They should be conveniently filed for reference at any time. For straight line charts suitable albums or scrap books make a convenient file. For round charts the scrap book form, though suitable and convenient, is bulky, as a page would be required for each day's chart. Substantial pasteboard boxes of the right size to hold a year's charts, are convenient. These can be labelled with the year, and, where several recording gages are in use, with the location of the gage, making it convenient to refer to the charts from any gage at any time.

It is interesting to look over the old gage charts, also often instructive. A new water works, with new and clean pipes, with pumps not over crowded, and ample size force and distribution mains should show a steady line, with slight fluctuations, and near the outer edge of the chart. As the consumption increases, the line grows less and less steady, and converges toward the center of the chart, showing the reduced pressure due to greater friction in the mains. The raggedness of the pressure line on the chart also increases with the age of the plant, as the mains become tuberculated and clogged with sediment. Finally, the consumption has gone beyond the pump capacity; a new and larger pump is installed, and the pressure line again approaches the outer edge of the chart, and becomes more steady, but still shows greater fluctuation than when the plant was new and working at a comfortable rate.

Notwithstanding the new pumping engine, the results are not entirely satisfactory, when compared with the earlier charts; the pumping engine, though of ample capacity, is working harder than the original pump had to work; both the water and steam pressure lines on the pumping station gages have approached nearer to the outside circles. Higher water pressure has to be maintained on the pumps to keep up the pressure on the distribution mains. A comparison of the gage charts from year to year shows this steady gradual increase of the pumping station pressures, and, at the same time, lessening of the pressure at the office and other points on the distribution system. Finally, the pumps are unable to give a proper pressure in the town, though they have a capacity even greater than the demand. A new force main is installed, and the original conditions are nearly restored, though the pressure lines are not quite as steady as at first, owing to bad condition of the pipe system. A thorough cleaning of the mains and distribution pipes restores the steady pressure line near the outer edge of the circle. So, the gage chart tells us when a new pumping engine is needed, when to install a larger force main and feed mains, when the mains need cleaning.

A study of the old gage charts tells a story, relates a plant history, as old documents or old letter files tell the story of the community. This is especially so if the charts are carefully and properly labelled before they are filed away; the date, the temperature and weather conditions, notes concerning fires, breaks in mains, engine or steam plant troubles, etc.

A frequent examination of the files of old charts should be made, as they tell an interesting and valuable story of the operation of the works, and give valuable hints for needed changes and improvements.

There is now an instrument for ascertaining from the pressure lines on circular charts, the style now mostly used, the average daily or weekly pressures. These results can be conveniently plotted, so as to show compactly the average pressures throughout a year, or many years. The plotting of two or more years on the same sheet, using different colored inks for the different years shows compactly the comparative pressures for the equal seasons of each year. Temperature and weather condition notations, would add to the interest of these charts. For instance, the month of February in one year shows a steady and good pressure line, while the same month of the next year shows low pressure and a ragged line. The temperature reference shows for one year a moderate temperature, for the other

steady cold weather with extremely low temperatures. Without the temperature record the great difference in pressure would be hard to explain.

Consumption records should also be included, to account for the probable steady decrease in average pressures from year to year. Like the February temperature record, the mid summer temperature and precipitation records would also tell their story.

Recording gages are useful in so many ways, give so much information and so good a record of the daily operation of the pumping plant and the condition of the plant, that they are indispensable, and no well regulated water works should be without them, and none can have too many. A superintendent cannot have accurate knowledge of the operation and condition of the plant under his supervision without using pressure recording gages. It is false economy to try to do without them. The cost of installation and maintenance is small, triffling, as compared with the benefits.

Pressure recording gages require little care, if daily charts are used, and they give the best records—daily changing of the charts, winding the clock, and filling the pen, are all that is necessary. The supply pipe should be blown out occasionally, to keep it clean, and the clock kept regulated, so that the time records will be accurate. This last item is important, should disputes arise, concerning pressures at time of fires, particularly in towns where an increased fire pressure is required. The rise of the pressure line in the chart should correspond to the instant with the record of the fire alarm. In this connection, some care must be exercised in placing the charts, to see that they start on the correct time line. Some troubles have been caused by carelessness in this, occasionally, for some unapparent reason, the gage chart does not revolve, though the clock is going. This may be due to careless securing of the chart, or to putting in two charts at once, the under one revolving all right, but not receiving the impression of the pen or pencil, the outer one slipping on the smooth surface of the lower one and remaining stationary.

THE WATER SUPPLY AT WILMINGTON, DELAWARE

By Edgar M. Hoopes, Jr. C.E.[1] and James M. Caird[2]

In Wilmington, as in many other cities of the country, the water supply was derived originally from a number of wells with wooden pumps, and although the city was founded in 1658 by a Swedish colony, it was not until a century later, namely 1796, that the first movement for concentrated action tending toward the establishment of a general system took place. The first resolution upon the subject of water supply was introduced in the borough council under date of December 31st of that year, and provided for a committee "to inquire of the inhabitants of the borough who owned pumps which stood in the street, whether they would be willing to give them to the corporation who would take care to have them kept in order."

This committee was continued from year to year and evidently fulfilled its mission, but four years later the growing importance of the borough demanded some better system and another committee submitted a report and an estimate for pumping the water from a spring on the hill at Third and Tatnall Streets, to Market Street.

The estimate of the above committee was as follows:

	£	s	d
835 feet of pump logs for mains, cost	57	8	½
Digging and hauling for mains, cost	7	1	10½
Cistern to contain thirty hhds	48	0	0
	112.	9	11

Owing probably to the extravagance of this estimate, the project failed to materialize, and no further progress was made until the year 1804 when the Wilmington Spring Water Company was incorporated.

[1] Chief Engineer of the Water Department. From notes of Capt. C. H. Gallager, Theodore A. Leisen and John A. Keinle, former Chief Engineers of the Water Department.

[2] Consulting Chemist and Bacteriologist for the Water Department.

This Company had the power to levy a sum of money on such persons as should use water from their works. They remained in existence, extending their plant from time to time, until the year 1810, when it seemed to be the desire of the citizens that the stock of the said company should be owned by the Borough. The plant was therefore purchased for the sum of $10,000, and this date marks the inception of the Wilmington Water Department, ranking it as one of the earliest municipal water plants of the country.

The growing population evidently proved a tax upon the spring water supply and the pumps about the streets, and from time to time it was found necessary to increase these works until they could no longer accommodate the demands of the citizens. On June 5, 1820 action was taken relative to securing a water supply from the Brandywine Creek, and a committee appointed to report the probable cost of such a project.

This committee recommended a plant that could take water from the southerly side of the Brandywine, east of King Street, and that the water be forced through iron pipes to a reservoir located at the junction of Tenth and Shipley Streets, and thence be distributed through iron pipes to the city. The original site selected is the location of the present pumping station and plant.

Definite action upon this report was evidently not taken at the time it was submitted, for from the records it appears that the reservoir was not built and the existing works were enlarged by introducing new cisterns.

In June 1827, the first definite project for the introduction of Brandywine water into the city mains was inaugurated by the purchase of the water rights and mill property on the Brandywine, belonging to Mr. John Cummins, and the construction of a reservoir on the square bounded by Market, King, Tenth and Eleventh Streets, which is the present site of the county court house. This work was completed in the fall of the same year and the first water was raised by the double acting pump to the new reservoir at midnight on November 15. This pump was operated by water power, the water falling on an overshot wheel 14 feet 6 inches in diameter, carrying forty buckets, each bucket containing 2 cubic feet of water when full. It was manufactured by Prosper Martin of Philadelphia and was similar to those installed for the Philadelphia works at Fairmount Park Station.

Owing to the fact that there was a storm of opposition to the plan, the committee, while fully impressed with the conviction that the pump would eventually do as they had calculated was somewhat doubtful of the success of its first trial, and kept the event from the knowledge of the citizens. The result was that at the time of testing the pump, it was known only to thirty or forty persons, among whom were the most steadfast and zealous friends of the plan. The record also states "that many of these failed even to tell their better halfs, and as a consequence great uneasiness was manifested by some of them at the nonappearance of their missing husbands at a late hour, and the services of friends were engaged to look them up."

It is interesting to note that when the pump failed to turn over owing to the fact that the gate was but partly raised, the omnipresent "I told you so" proclaimed the project a failure, but when the full head of water had been turned on the ponderous wheel revolved, and the double action pump was set in motion.

From a historical standpoint the following quotation from *The American Watchman* of December 4, 1827, may be of some interest.

NEW WATER WORKS

According to promise we lay before our readers some further particulars respecting the works just completed for supplying the Borough with water. For most of the details, we are indebted to the politeness of Jonas P. Fairlamb, Esq., Civil Engineer, whose professional services were engaged by the Corporation for this important work and it affords us pleasure to state that the result of a trial of the machinery has proved the accuracy of his calculations and borne honorable testimony of his talents as an engineer.

The capacity of the original reservoir, as supplied by this pump, was 1,000,000 gallons, and was enlarged as the population of the borough increased until the demand for better pressure in the outlying districts necessitated the construction of a reservoir at a higher elevation. This demand was satisfied by the installation of the Rodney Street reservoir (now abandoned) in the year 1863. Later, 1873–77 the inadequate storage capacity of the low service reservoir was augmented by the construction of Cool Spring reservoir, this having a capacity of 40,000,000 gallons.

No further changes were made in the present system until recent years, and the department remained under the direct control of the borough and under the management of a water committee until 1833,

when the state legislature, at the request of a number of citizens, created the board of water commissioners to whom was entrusted the care and management of the water and the improvement of a water supply system.

THE EXISTING SYSTEM

The source of supply has not been changed, the water being diverted from the Brandywine Creek, at a dam approximately 4800 feet up the stream, from the main pumping station, to which it is conducted by a race on the south side of the Creek.

Originating in the lower counties of Pennsylvania, the Creek has a total length of approximately 35 miles, with a drainage area of 335 square miles above the city dam. Owing to the number of small towns and villages, together with various classes of manufacturing plants, located on the watershed, the water is contaminated to a marked degree. In recent years the Board of Water Commissioners of the City, assisted by the State and City Boards of Health, have endeavored to remove the principal sources of pollution, and, while this has been accomplished to a considerable extent, by a rigid enforcement of the laws, and regulations governing the pollution of the stream within our own State—it has been impossible to secure any satisfactory results in Pennsylvania, which controls two-thirds of the watershed, and where the gravest dangers from sewage pollution exists.

Table showing typhoid fever deaths and death rate before removal of some sources of pollution

YEAR	DEATHS	RATE
1900	36.0	47.0
1901	32.0	41.0
1902	42.0	52.0
1903	54.0	66.0
1904	43.0	51.0
Average five years	41.4	51.4

The sanitary work on the watershed commenced in 1905 several sources of pollution being removed, following which there was a reduction in the typhoid fever death rate.

Table showing typhoid fever deaths and death rate after removal of some sources of pollution

YEAR	DEATHS	RATE
1905	32.0	38.0
1906	34.0	40.0
1907	30.0	35.0
Average three years	32.0	37.6

These results show that following improvements on the watershed there was a reduction of 26.9 per cent in the typhoid fever death rate.

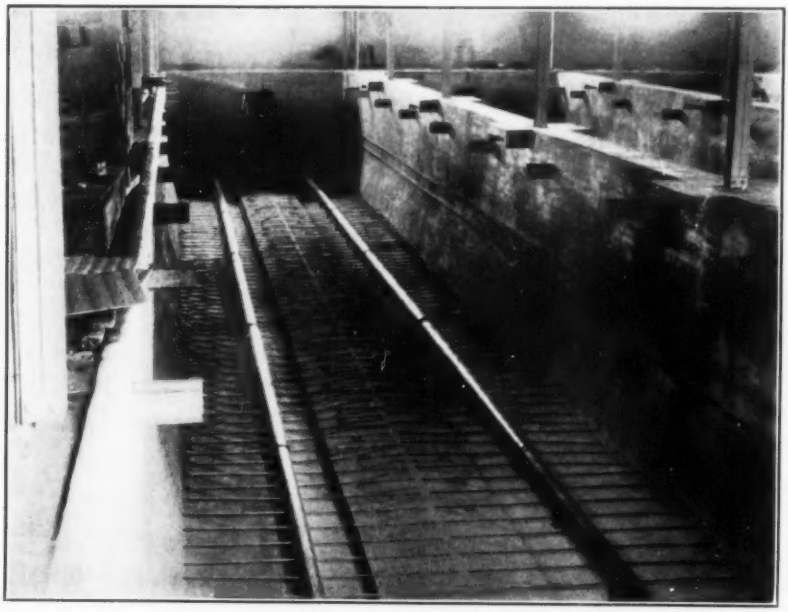
In the year 1892, the Board installed a purification plant known as the Sellars filter, and this, until early in 1903, had been supposed to be giving excellent service. Such however was not the case, as a thorough physical and bacteriological investigation clearly indicated.

A movement towards securing a more efficient system of supply, was therefore inaugurated early in the summer of 1902, but this was looked upon primarily as an extension to the then present system, and was brought about principally to relieve the precarious condition on the high service, or the Rodney reservoir.

The growth of the city has been such that the capacity and elevation of this reservoir were insufficient, and furthermore its length of service had rendered its condition not only treacherous but dangerous to public safety. A site was selected and purchased late in 1902 for the construction of a larger reservoir at a higher elevation, but legal obstacles delayed the beginning of the work until the fact that the old filter was of no benefit as a purifier was brought into the lime light, and the purification of the city's supply was agitated, discussed and finally decided upon. In the solution of this problem, the troubles of the department began. They were many and of a varied character, including the recommendation of new sites, suits in court, arbitration proceedings and principally financial embarrassment. One by one they were overcome, and by piecemeal the work has reached its present state. There have been to date ten major contracts awarded and completed which include the following: The supplying and laying of approximately 9000 linear feet of 43 inch lockbar, steel force main; 10,500 feet of 48 inch lockbar, steel distributing main; two 12,000,000 gallon Holly vertical, triple expansion pumping engines; four 300 h.p. Edge Moor water tube boilers and accessories; the construction of a large pumping station, boiler house;

35,000,000 gallon storage reservoir; a preliminary filter of 15,00,000 gallon daily capacity; a reinforced concrete seven arch bridge and aqueduct, 363 feet long; and the construction of a slow sand filter plant of the intermediate type, having a daily capacity of 15,000,000 gallons, including a 6,000,000 gallon filtered water reservoir.

The water passes through the diversion canal or race, to the preliminary filters, thence by gravity to the pumps and is raised by them to the new (Porter) reservoir. After approximately three days of



AIR WASH MANIFOLD OF ONE PRELIMINARY FILTER BED

sedimentation it flows by gravity on to the final filter beds, thence to the filtered water reservoir, and finally to the 48-inch steel distribution main supplying the high service system direct and the low service (Cool Spring) reservoir.

THE PRELIMINARY FILTERS

This plant consists of ten beds of 14 feet 5 inches by 100 feet and a reservoir 105 feet 9 inches by 30 feet 4 inches inside dimensions

with a depth in each of approximately $8\frac{1}{2}$ feet. The construction of the foundations and filter walls was of reinforced concrete, the superstructure being of brick, with limestone trimmings. After drawing the water from the race, it is fed to the beds from an influent gallery and, passing under the filter medium, through the inlet drains, is distributed by them over the area of the bed. The system of filtration is upward, the water rising from the drains through 6 inches of gravel of various sizes, thence through 20 inches of small coke, and finally through 20 inches of sponge clippings, the later being held in place by cypress racks. After passing the bed, the water is delivered into an effluent gallery and conveyed thereby to the clear well.

The distinguishing feature of this plant is the method employed in cleaning the filter medium. Unlike other plants of its type (in which the sponges are removed) the washing of the coke and sponges is done directly in the beds with air and water.

For this purpose, there is installed in the bed under the filter medium, a system or manifold of air pipes, consisting in each bed of two 6-inch main headers with $\frac{1}{2}$-inch laterals at 12-inch intervals. The latter pipes are plugged at the end and perforated with $\frac{1}{16}$-inch holes on 3-inch centers. The flow of air is secured from a rotary blower, and is discharged through the bed under about 5 pounds pressure. Water is discharged through the bed under normal operating heads in either direction, up or down, and the dirty or wash water is carried off by suitable connected drains. This system of cleaning the sponge clippings and coke has been found not only satisfactory, but very effective, and is carried on at a greatly reduced cost of operation and maintenance, only one attendant being required at all times.

The preliminary filters were placed in operation in October, 1907, two years before the final or slow sand filters.

Table showing average bacteria per cubic centimeter raw and pre-filtered waters, Wilmington, Delaware

YEAR	RAW	PRE-FILTERED	PERCENT REMOVED
1908	4306	2023	53.02
1909	2916	1448	50.35
Average two years	3611	1736	51.69

These filters are operated at the rate of about 50,000,000 gallons per acre per day.

Table showing average turbidity, in parts per million, raw and pre-filtered waters, Wilmington, Delaware

YEAR	RAW	PRE-FILTERED	PERCENT REMOVED
1908	26.0	9.0	63.08
1909	39.0	15.0	61.53
Average two years	32.5	12.0	62.35

The turbidity removal was slightly higher than the bacterial.

Table showing deaths from typhoid fever and death rate, two years operation of pre-filters, Wilmington, Delaware

YEAR	DEATHS	RATE
1908	21	24.0
1909	25	27.0
Average two years	23	25.5

Table showing average deaths from typhoid fever and rate, three years before and two years after filtration

	DEATHS	RATE
Before	32	37.6
After	23	25.5

These results show that after the prefilters were in operation the typhoid fever death rate decreased 32.2 per cent.

THE PUMPING STATION

The interior dimensions of this building are 60 by 110 feet by 54 feet the last dimension being the height for two floors. The interior basement walls are of Kittanning brick, while above the main floor the walls are of porcelain faced brick wainscoted with white enameled brick and a brown enameled brick baseboard. As noted before, there are installed two 12,000,000 gallon engines, and there is space available for the installation of a larger unit. The exterior walls of the station are of light buff, mottled brick, with granite and terracotta trimmings.

THE SEDIMENTATION RESERVOIR

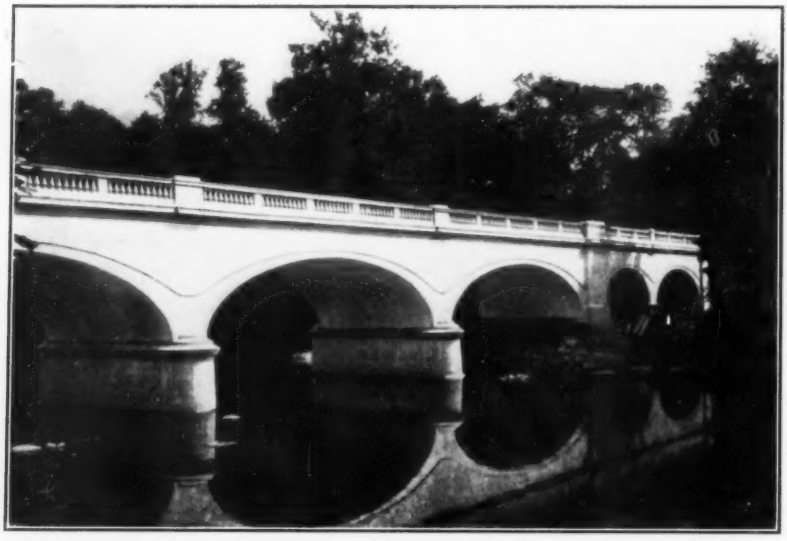
The total capacity of this reservoir is 35,000,000 gallons of which 32,000,000 are available. It is located on the 89 acre tract of land originally purchased by the Board for its construction and is distant from the pumping station about $1\frac{3}{4}$ miles. It is triangular in shape, with rounded corners, and is located with reserve space for a similar reservoir, when future extension is required. The maximum elevation of water is 285 feet above mean low tide, or city datum, and the

FILTERED WATER RESERVOIR (UNDER FILTRATION PLANT)

depth of water ranges from 21 feet over the sump to 17 feet at the inside toe of the embankment. The inner slope of this embankment is $1\frac{1}{2}$ to 1 and is paved with concrete and Belgian block upon a lining of 12 inches of clay puddle. The bottom is all concrete, laid on a bed of 12 inches of puddle. The exterior slopes are variable, reaching a maximum of 2 to 1. Around the top of the embankment there is an 8 foot macadam roadway, with a 2 foot cement walk and a Scotch coping rubble wall 3 feet high. The 43-inch forcing main discharges the water to the reservoir through four branches at a point 400 feet from the apex of the triangle. At the apex, the water is drawn off

over sluice gate weirs constructed in a tower and is thence delivered to the final filters. This tower is so constructed that its upper story serves as a pressure tank for local supply about the plant.

In the laying of the large mains numerous difficulties had to be overcome, many of which required change of alignment, after the pipe had been designed and purchased. While the majority of this pipe was of the lockbar type, there were occasional instances where short radius bends required the use of ordinary rivetted steel construction.



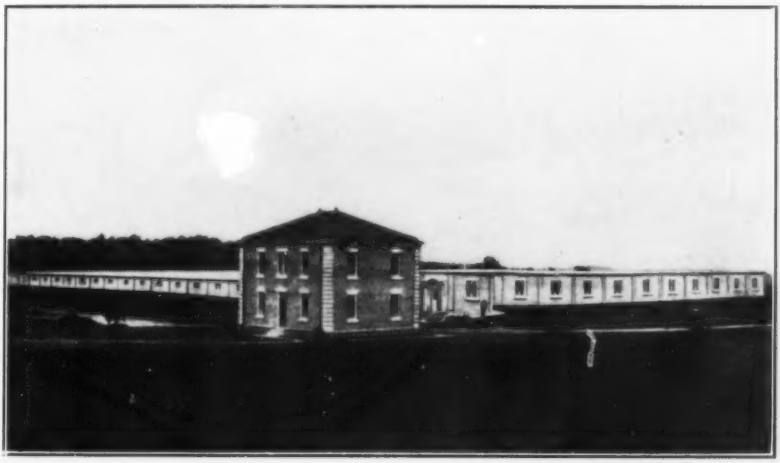
Van Buren Street Bridge and Aqueduct

VAN BUREN STREET BRIDGE

It was originally intended that the distribution main should be laid under the bed of the Brandywine Creek, at its crossing at Van Buren Street, but, after some discussion, it was decided to build a bridge for this purpose, and also to accommodate the traffic between the parks on each side of the stream. To this end the cost of the work was borne two-thirds by the Water Department and one-third by the Park Commission. The structure is of reinforced concrete of the Melan design and consists of three main spans of 56 feet and four approach arches, three of which are 28 feet and one of 33 feet span.

FINAL SLOW SAND FILTRATION PLANT

Radical changes were made in the design of this plant, these changes involving a type of construction which is an innovation in this character of work, no similar plant having been constructed elsewhere. After much legal discussion together with financial difficulties, the contract for this work was finally executed on December 9, 1908. Though the construction work was not entirely completed within contract time, it has been possible to operate the filter, and since January 16, 1910, filtered water has been distributed throughout the city.

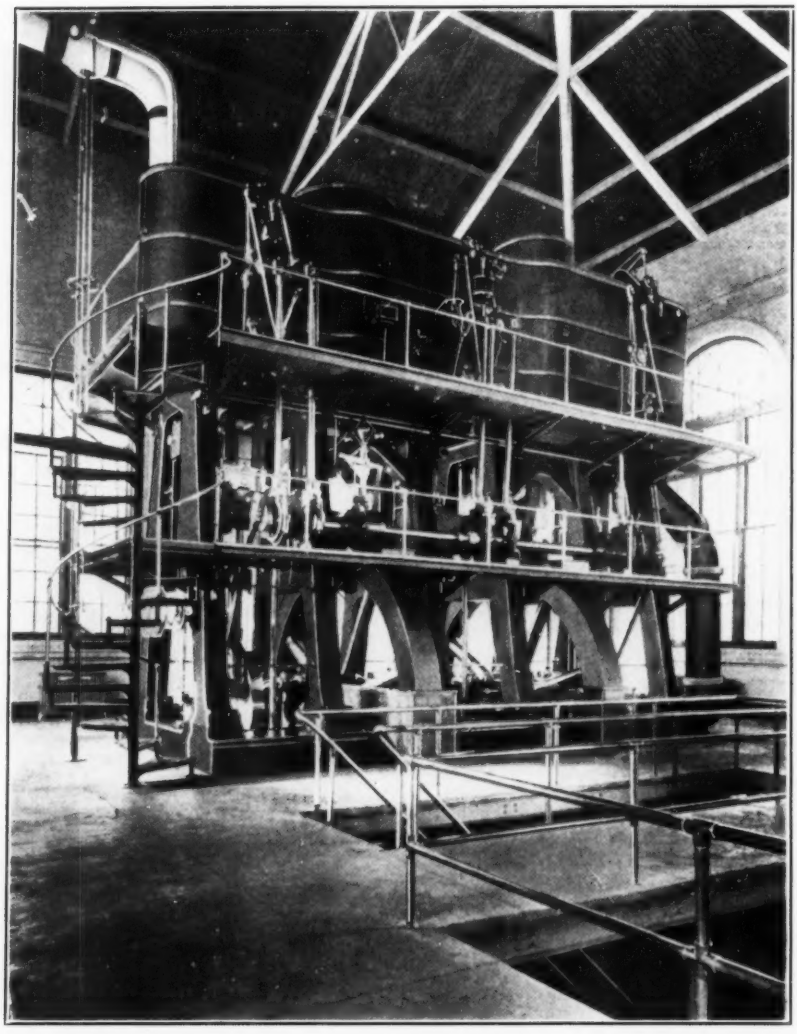


Administration Building and Slow Sand Filtration Plant

The plant consists of six covered sand filters, of one-third acre each in area, a filtered water reservoir, a sand washing machine of the Blaisdell type, in addition to a control house, Venturi meters, valves and piping, and was designed for a daily capacity of 15,000,000 gallons, although for present needs a 12,000,000 gallon capacity is ample.

The six filter units are superimposed upon a reservoir of 6,000,000 gallons capacity. At the extreme end of the works a dry gallery runs across the width of the six beds which contains the influent and effluent pipes and regulating devices, all visible and accessible.

One of the principal features in the design is that the filters and filtered water reservoir are combined in one structure two stories



12,000,000 Gallon V. T. E. Holly Pumping Engine (in Duplicate)

high. The reservoir or lower story has a groined arch roof supported on square piers, and this roof serves also as the floor of the filters, over which extends a roof of concrete slabs supported on I-beams. This form of construction effected a considerable saving in concrete and in excavation. If the two parts had been built separately a roof would have been necessary for each. There is also the obivious saving in land area. With such a grouping of the parts there is, of course, a far greater loss of head than in the case when the structures are built separately. Local conditions had a large influence in the selection of this type of structure, for the works are on high ground, and the loss of head between the filters and the reservoir is not a matter of any great consequence.

Another unusual feature is the shape of the beds, which are long and narrow; they measure 362 by 40 feet, and were built of this size in order to use a sand washing machine.

The system of underdrains also is somewhat unusual, for the filter effluent collected by a tile pipe drain passes through holes in the floor into a discharge pipe hung up close to the roof of the filtered water reservoir.

This reservoir measures 364 by 252 feet inside, and has a plain concrete floor of inverted groined arches. The piers, which are spaced on 14 feet centers, are 24 inches square and $6\frac{1}{2}$ feet high from floor to the springing line of the groined arch vaulting. The minimum thickness of floor and roof is 9 inches.

The filters which form the second story of the concrete structure are separated from one another by concrete walls, which are placed over every third pier across the width of the plant. These walls are 24 inches thick and 9 feet high, and along their side are bolted cast iron brackets, which support rails for carrying the trucks of the sand washing machine. In the top of each wall a channel is cast and will serve to convey the dirty wash water from the washer to the waste pipe connected to a sewer. The underdrainage system consists of one row of Brossman tiles, laid longitudinally down the center of each filter. These tile drains are 2 feet long and have a section resembling a flat inverted V. Water may enter these through four openings, two on each side. They are bedded to the floor by a coat of grout, and are covered with a 15-inch layer of broken stone.

Under the tile pipe collectors at 14 feet intervals along the longitudinal axis of each filter compartment holes are cast in the concrete floor and extend through to the reservoir below. The purpose of

these holes is to afford an outlet for the filtered water, and at the same time serve as a means of supporting the effluent pipes, which are suspended under the roof of the filtered water reservoir, and extend the long way of each bed, running directly beneath the main tile collectors. The effluent pipes are of cast iron, increasing from 6 to 20 inches in diameter. The effluent piping suspended at 14 feet intervals for its entire length, connects with a main effluent pipe in the dry gallery at the end of the plant.

The lower courses of the filtering material are 14 inches of crushed stone and graded gravel, and on them rests a layer of sand 24 inches thick. The sand was dredged from the Delaware River and had an effective size of 0.23 mm. and a uniformity coefficient of 1.83. The head of water on the sand is $4\frac{1}{2}$ feet, and the beds are operated at a nominal rate of 6,000,000 gallons per acre daily for the present, although it is believed that a rate of 8,000,000 gallons can be used by taking precaution to sterilize the filtrate.

The roof of the filters is of concrete reinforced with triangular wire mesh. It is laid in 3 inch slabs upon 8 inch I-beams, and supported by I-beam columns, resting on the walls dividing the filter beds. The roof has a slope from the center toward either side of $\frac{1}{4}$ inch in 1 foot.

At the southwest end of the plant, across which the dry gallery extends, the roof had to be raised a few feet in order to afford clearance for the sand washing machine when it is transferred from one bed to another. A continuous bracket cast in each gallery wall carries the rails, along which a transfer carriage travels. This carriage carries a pair of rails spaced 38 feet apart, the same gauge as those of the filter beds, so that the machine may pass from the beds to the transfer carriage and then be carried by it transversely to any position in the gallery which may be desired.

At the northwest end of the pipe gallery there is a valve chamber to control the head of water on the beds. Water enters it by gravity through a 48-inch and 30-inch pipe from the sedimentation reservoir, which ends with a 30-inch hydraulic and hand operated disc valve originally intended to be operated by a float in the chamber. By a simple and occasional adjustment of the hand wheel the water is maintained at a uniform level on the filters. From the valve chamber the water enters an influent main running the length of the gallery, reducing in size at each bed from 30 to 16 inches in the total length of the gallery. This main has a 16-inch branch at each bed, and

water from it discharges into a channel along the end of each filter and flows over the inner edge, which acts as a weir, and down through the sand. The filtered water after passing through the pipes suspended from the roof of the filtered water reservoir, discharges through a Venturi meter on each bed, into the main effluent collecting pipe and thence into the filtered water reservoir.

The final or slow sand filters were placed in operation January 16, 1910.

Table showing average bacteria per cubic centimeter raw, pre-filtered settled and final effluent waters

YEAR	RAW	PRE-FILTERED	SETTLED	FINAL EFFLUENT
1910	17,517	8,261	16,177	1,012
1911	36,546	18,507	17,056	146
1912	36,948	24,855	20,257	491
1913	18,042	12,448	10,009	207
Average.....	27,263	16,025	15,8 75	464

Bacteria removed by,	*per cent*
Pre-filters	41.23
Settling basin	00.94
Final filters	97.08
Total	98.30

Table showing average percent of positive presumptive tests for B. Coli-communis, in 1 cc. samples, raw, pre-filtered, settled and final effluent waters

	RAW	PRE-FILTERED	SETTLED	FINAL EFFLUENT
	per cent	*per cent*	*per cent*	*per cent*
Average.....	97.9	93.4	90.9	11.2

B. Coli-communis removed by,	*per cent*
Pre-filters	2.56
Settling basin	2.68
Final filters	87.68
Total	88.52

The general bacterial removal was higher than that of the B. Coli-communis.

Table showing average turbidity, in parts per million, raw, pre-filtered, settled and final effluent waters

YEAR	RAW	PRE-FILTERED	SETTLED	FINAL EFFLUENT
1910	52.0	19.0	16.0	0.0
1911	60.0	27.0	15.0	0.0
1912	85.0	49.0	43.0	2.0
1913	72.0	53.0	37.0	0.1
Average.....	67.2	37.0	27.8	0.8

Turbidity removed by,	*per cent*
Pre-filters	44.95
Settling basin	24.87
Final filters	71.23
Total	98.82

The total bacterial removal is about the same as the total removal of the turbidity.

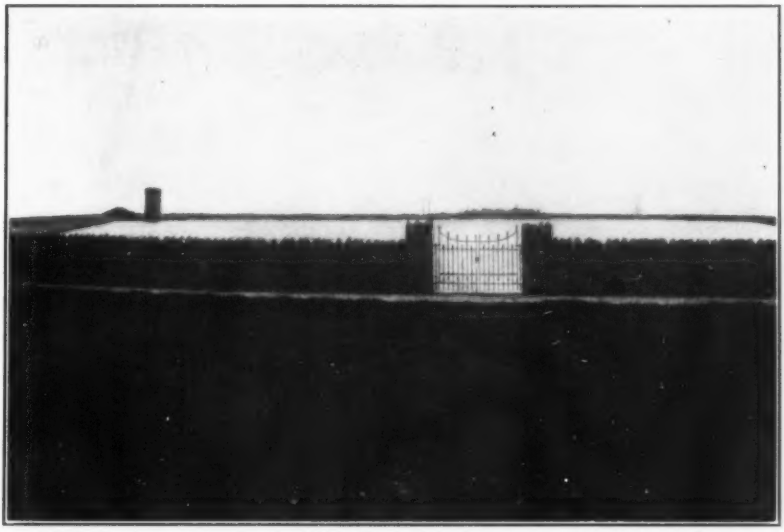
As a large portion of the turbidity is removed by the prefilters, there is little work for the settling basin to do, the size of the particles passing these filters being very small, the amount that settles is very slight. There is no mechanical action to assist in carrying down the bacteria, hence the removal by the settling basin is small.

The color of the water has been determined only during the past two years.

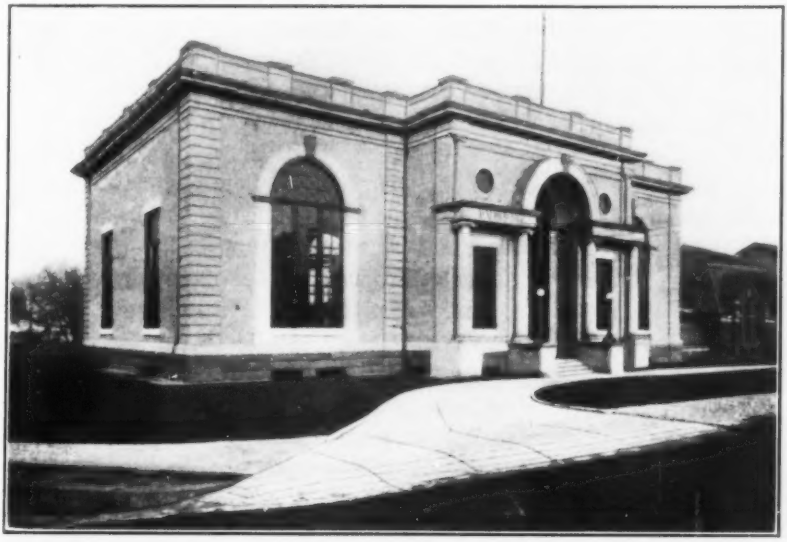
Table showing average color, in parts per million, raw, pre-filtered, settled and final effluent waters

YEAR	RAW	PRE-FILTERED	SETTLED	FINAL EFFLUENT
1912	25.0	23.0	24.0	11.0
1913	21.0	21.0	20.0	6.0
Average.....	23.0	22.0	22.0	8.5

Color removed by,	*per cent*
Pre-filters	4.35
Settling basin	0.00
Final filters	61.37
Total	63.05



WM. T. PORTER RESERVOIR



BRANDYWINE PUMPING STATION

As no coagulant is used, the removal of the color is not high or complete.

The final effluent during eight months of 1913 was treated with liquid chlorine.

Table showing average bacteria per cubic centimeter and per cent of positive presumptive tests for B. Coli-communis in 1 cc. samples, before and after liquid chlorine treatment

BACTERIA PER CC.		B. COLI-COMMUNIS	
Before	After	Before	After
		per cent	*per cent*
302	51	11.97	2.77

These results show that the liquid chlorine removed 83.12 per cent of the bacteria and 76.86 per cent of the B. Coli-communis.

The amount of liquid chlorine applied averaged 1.56 pounds per 1,000,000 gallons.

During January 1912, there was an epidemic of typhoid fever at Coatesville, Pennsylvania (on the watershed). No notice of this condition was sent to the water department until the epidemic was well under way, and before an emergency hypochlorite plant could be installed and placed in operation the filters had failed to remove the bacteria and several cases of fever started in Wilmington.

When the condition of the water-shed was learned an emergency hypochlorite plant was placed in operation, the hypochlorite being added to the water after it left the settling basin and before it was upon the final filters. This treatment killed the bacteria in the water and also those on the beds, and it was several months before the beds returned to normal working conditions. In the mean time a more permenant hypochlorite plant was placed in operation and the final effluent was treated.

The use of hypochlorite prevented a serioûs outbreak of typhoid fever in Wilmington.

After this experience, experiments were made with liquid chlorine, the results being so satisfactory that a permenant plant was installed and during 1913 it was operated eight months.

Table showing typhoid fever deaths and rate since completion and operation of final filters or total plant

YEAR	DEATHS	RATE
1910	32.0	36.0
1911	24.0	27.0
1912	24.0	26.0
1913	11.	12.0
Average	22.7	25.2
Average for two years when only pre-filters were in operation	23.0	25.5

The average results show that the typhoid fever death rate did not decrease after the final filters were placed in operation.

The results do show that during the year 1913, when the liquid chlorine plant was in operation that the typhoid fever death rate was 12 per 100,000 which is 53.9 per cent less than during the previous year.

MECHANICAL WASHING OF FILTER SAND

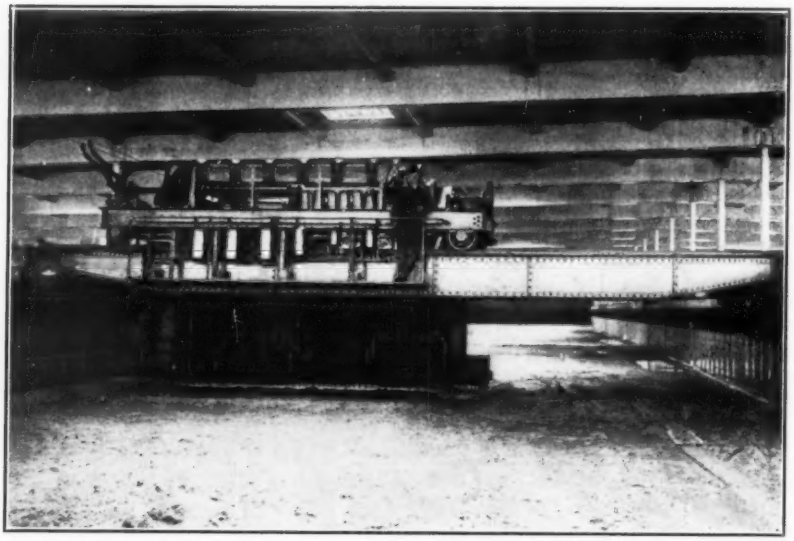
The Blaisdell mechanical washing of filter sand is a method of recent introduction, and in consequence is not as generally known or understood as those systems and methods that have been heretofore applied for filter cleaning. Briefly, the washing machine may be described as a traveling crane spanning a filter unit and supporting a watertight rectangular chamber containing the essential washing machinery, and further provided with means of lowering this chamber to the sand surface and traversing over the filter while the operation of washing the sand is proceeding. The washing chamber may be raised so as to clear the rim of the filter, and the machine removed to another unit by the transfer carriage.

As the width of each filter bed is forty feet, the span of the main traveling crane supporting the washing chambers was made thirty-eight feet on rail centers. Forty pound rails were used, supported on specially designed cast iron brackets bolted through the division wall on $2\frac{1}{2}$ feet centers.

In the design of the machine the water tight washing chamber was made 20 feet long by 4 feet wide by 5 feet 6 inches deep. Approximately 2 feet above the sand level or bottom of this chamber there is a plate or diaphragm subdividing the washing chamber into two compartments: the lower known as the suction chamber, from which

the dirty wash water is withdrawn, contains five horizontal stirrer wheels supported on vertical shafts; the upper, or dry chamber contains the driving mechanism for the stirrer wheels and also the pressure and suction pumps.

As the width of the bed is 40 feet, and the area washed in the forward movement of the machine is but 20 feet, the entire washing chamber is supported upon crane beams mounted upon wheels, for the purpose of moving it laterally across the width of the bed, on rails supported on the girders of the main traveling crane. This permits of com-



FILTER SAND WASHING MACHINE

pleting the washing of the entire bed in one round trip of the machine, i.e., by moving forward over the entire length in one position a width of 20 feet is cleaned and by transferring the chamber to the opposite end of the main crane, which transfer is made without stopping washing process, the remaining area is cleaned upon the return trip, leaving the entire machine ready to be transferred to any bed without undue loss of time.

The stirrer wheels in the lower or suction chamber are supported by vertical shafts in a horizontal position 1 inch above the sand surface. Numerous teeth project from the lower face of the stirrer wheel rim

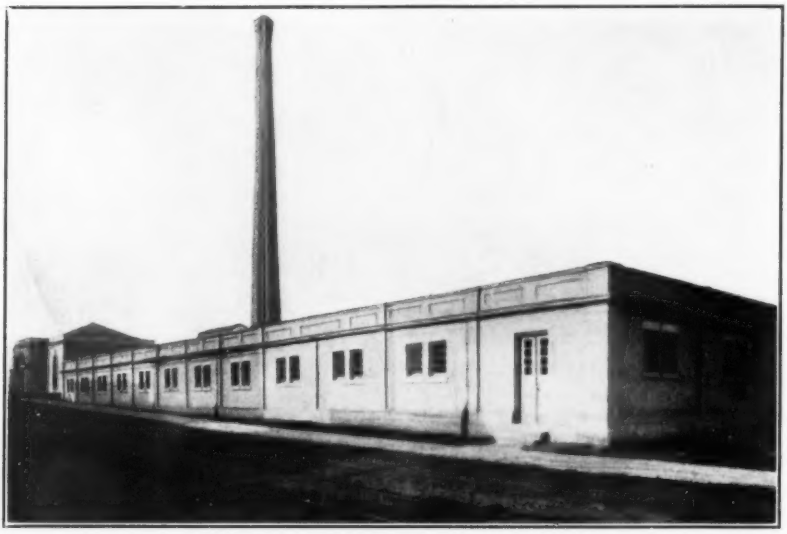
and extend into the sand below the level of the chamber to any desired depth. Each vertical shaft passes through a stuffing box in the top of the chamber to an upbearing in the frame of the machine, and is revolved by a motor actuated worm gearing at the upper end. Midway of the shaft there is a T-shaped sleeve, with stuffing boxes at the upper and lower ends, which serves as a water connection. From this sleeve downward each shaft and wheel including spokes and rim are hollow. The teeth are hollow and perforated for the purpose of creating a water jet action from the supply delivered by pressure pump. Water supply for the pressure pump is taken from the raw water of the filter, while at the same time a suction pump connected to the top chamber withdraws not only all the water the pressure pump supplies through the perforated teeth, but also an additional quantity made up from the filtered water stored in the filter sand. In operation, the teeth scour the sand; the wash, or pressure water, by the jet action prevailing, drives the dirt up into the suction chamber; the clear water stored in the filter bed is drawn into the washing zone by the excess suction over pressure supply, and the wash water is withdrawn and pumped from the chamber before the machine passes a given point. The chamber is sealed to the sand surface by wide hinged shoes extending in advance and to the rear of the front and back plates, while the side plates form a cutting edge extending down into the filter sand.

The maximum speed of operation is governed by the rapidity with which the pressure pump water may be displaced by the filter bed supply, and as a result the speed of advance varies with the length of teeth used, or in other words, the depth of the washing.

Due to the fact that there is less dirt at the lower depths of the filter than near the surface, it is necessary when washing to a considerable depth such as 24 inches, that teeth of proportionate length be fixed in the stirrer wheel, and the wash water is therefore introduced not only at the lowest point, but at several points above. This pressure water quickly rises through the disturbed sand zone and is displaced by the inrush of the clear water, the upward current of which occurs well toward the center of disturbance created by the teeth and covered by suction chamber. The sand grains are dislodged and forced apart by the passage of the teeth, and as they return to the cavity in the wake of the teeth the strong upward current of wash water creates a temporary suspension and churning action within the area of the suction chamber. During this time the dirt

and light particles are swept to the surface and withdrawn with the wash water. The subsidence of the suspended sand takes place following the passage of this violent upcast and occurs during the more gentle upward passage of the clear water, so that when the sand comes to rest it is uniformly water packed and free from all air.

As each stirrer wheel revolves five times per minute and mounts thirty-two teeth, it follows that all sand grains are disturbed several times, both by direct contact with the teeth as well as by the working of adjacent sand and the horizontal water jets, from the teeth.



PRELIMINARY FILTER BUILDING

The washwater may be controlled to enable hydraulic sizing if so desired, working all very fine sand to the surface, and if sand too fine for effective size is present, it is possible to remove it from the filter by increasing the duty of the pressure and suction pumps to secure the desired upward velocity to hold the sand in suspension. At all times the excess suction demand is satisfied by the supply of clear water stored in the filter sand.

The dirty wash water from the machine is discharged to a gutter formed in the concrete party wall between the filter sand units, from which there are sewer connections at either end. This gutter is about 18 inches deep, 10 inches wide at the top and 6 inches at the

bottom, with a slight grade either way from the center of the beds to the drains at the ends, but the slope is not enough to allow any sand raised to proceed to the sewer system. Sand that collects in the gutter is removed with a spade and returned to the filter or taken away as desired.

The party walls serve to support the brackets for the rails on which the washing machine travels, and also the steel columns supporting the roof over the plant. The wash water gutter is placed off the center line of the wall to enable the discharge spout from the machine to pass the columns.

All of the several operations of the mechanical features of the machine are actuated by independent motors controlled from the platform by one operator. There are six motors mounted on the machine with variable speed controllers for operating as follows:

	CAPACITY H.P.	MINIMUM H.P.	MAXIMUM H.P.
Main traveling crane	25.0	5.0	15.0
Washing chamber transfer	7.5	1.5	2.5
Washing chamber hoist	7.5	5.5	5.5
Stirrer Wheel drive	20.0	5.0	20.0
Pressure pump drive	25.0	12.0	15.0
Suction pump drive	10.0	3.0	5.0
	90.0	32.0	63.0

Actual experience has demonstrated that but 54 horse power is required for the worst conditions of operation, and 25 horse power with short teeth and normal conditions. The quantity of current consumed in washing each bed of one-third of an acre in area has ranged from 80 to 175 kilowatt hours, and averages throughout the year approximately 120 kilowatt hours. The current consumed and power required is least when the shortest teeth are being used and reaches a maximum with the longest teeth during the winter months, when the oil gear boxes tend to congeal and the operation of the machine is hampered by ice on the filter beds.

With a five unit machine cleaning a strip 20 feet wide, one operator washing one-third of an acre in four hours, including transfer, and the bed is usually back in service within five hours from time of closing valves for washing process. This question of time element naturally effects the reserve area of plant that must otherwise be constructed

in order to maintain normal rates of filtration throughout. The experience of the Department has been that the time out of service by this method per acre area ranges from 7.5 per cent to 10 per cent, while with the old method of hand scraping, ejecting, and restoring, from 15 per cent to 25 per cent of time is lost. It is apparent therefore, that the construction cost, either for reserve area or plant capacity for low rates of filtration, is materially reduced by this improved mechanical method of filter cleaning.

The method also had a material effect upon the cost of operating sand filters, effecting a reduction in our plant of approximately 50 per cent over the cost of operation of plants of the same capacity elsewhere, notably Philadelphia.

It was confidently expected when the plant was built that the operating cost would not be greater than $1 per 1,000,000 gallons, exclusive of laboratory charges. We have not, however, been able to reach this low figure, the cost in the year 1910–11 being $1.08. As the operations of the plant require intelligent superintendence at all times, and as skilled mechanics must be employed continuously for operating the washing machine, notwithstanding that it is idle approximately 75 per cent of the available working time, it is evident that with a larger daily delivery in million gallons, the above unit cost which involves a considerable overhead or fixed expense, could be materially decreased.

When the turbidity of the settled or applied water is above 50 parts per million, it is impossible to get a filtrate that is entirely satisfactory.

At one time, during cold weather, it was necessary to use dynamite to dislodge the ice from the filters.

The most serious trouble is from "air bound" beds, during the winter months, when the water is saturated with oxygen.

These slow sand filters produce results equal to other plants of the same type while the troubles are no greater.

The installation of the liquid chlorine plant has proved to be a very valuable addition.

THE CROTON WATER SUPPLY: ITS QUALITY AND PURIFICATION

By George W. Fuller

It may seem curious that the writer should present to you, at this first meeting of the new section of the American Waterworks Association, a paper on a dead project. But this particular dead project is of much significance, both because of its local interest and because it in some ways marks a period in the history of the improvement of American water supplies.

During the last few years the Croton water has been the subject of much controversy which has been fully reported in the technical papers. But, while the writer gives to this controversial aspect the space its importance warrants, he is writing this paper primarily to bring before the engineering profession the technical side of the project and to present certain data which have not yet been made public.

It has often been remarked that we in America lag far behind our European contemporaries in those things which are often taken as an index of an older civilization. Particularly are we backward in those activities which are governmental functions. We take it as a matter of course that our police should find it difficult to repress crime and our courts to punish it, after it is committed; that our fire losses should be vastly higher than are experienced abroad; that billboards should deface our streets for the profit of private individuals; that our sewage waste should be allowed to become a nuisance to our neighbors.

And with that, though we have begun to have a little better appreciation of the necessity of a clean drinking water supply, our standard is still far lower than is accepted abroad. In Germany it is an unwritten law that all surface supplies, no matter how well protected, must be filtered. In England this practice is almost as common. For instance, the water supply for Birmingham, coming from the Elan Valley, 80 miles away in the Welsh mountains, from a 71 square mile watershed containing no villages and only two farms with occasional flocks of sheep, is filtered; likewise the Liverpool supply, coming 68 miles from the Vyrnwy, from a watershed of sim-

ilar character is filtered. And it is noteworthy that such filtration is considered necessary, not because any tests indicate a danger of infection, but because a surface supply is always liable to pollution from unexpected sources. And later developments in the art of water purification still leave unchanged the condition that only filtration is an assured protection against water pollution.

Time was when our New York supply was in the front rank of American waters. That position has long since been lost; not because the Croton water is getting worse than it was, but because our standard is rising and our American communities are expecting water better in appearance, palatabilty, and in genuine safety.

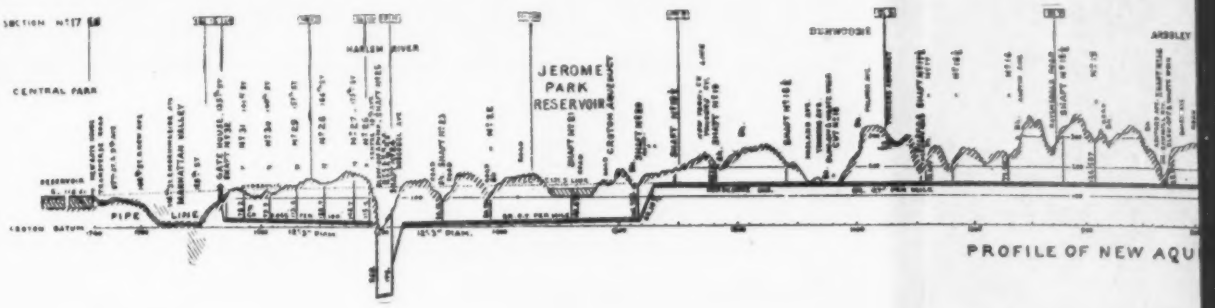
It is to the credit of the engineering and medical professions that they have in and out of season reiterated this need. And they do not have to, nor do they try to, base it on a showing that the water is at present dangerous. Only recently our Commissioner of Health, Dr. E. J. Lederle, testified before an investigating board that, quite apart from any showing of chemical or bacterial analysis or of any typhoid statistics, he considered it proper that our water should be filtered.

The project which has recently failed will unfortunately mean a lapse of some years before the public will realize its needs. It will interest you, however, as an engineering question, and it will form a basis for an instructive comparison, some years hence, when filtration will be finally effected for the Croton water.

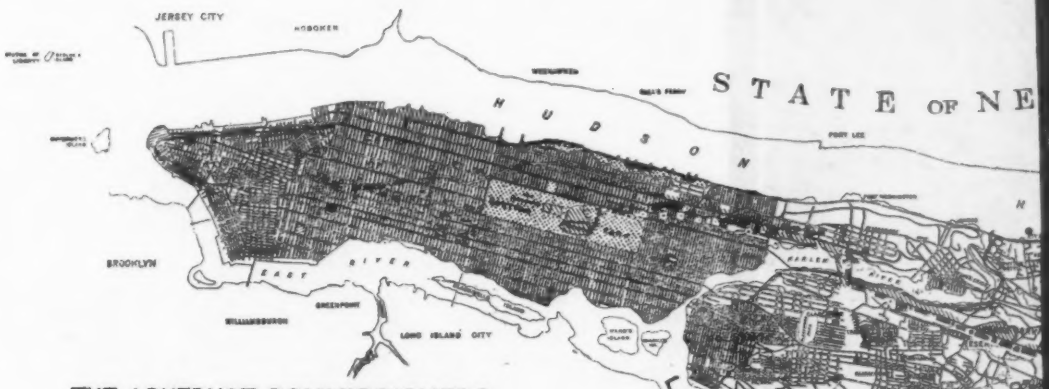
THE WATERSHED

The bulk of the water supply for the Boroughs of Manhattan and the Bronx, of the City of New York, comes from the Croton watershed, located roughly 30 miles north of 135th Street. The supply generally drawn from this source is about 300,000,000 gallons per day. The location of the watershed and the aqueduct connections to the city, are shown on the map on Plate I. On this map are shown the various artificial lakes forming the water storage, and the connection to the Central Park reservoir through the two aqueducts.

The watershed is a cultivated agricultural section, with moderate slopes. The total area tributary to the water supply reservoirs is 360.4 square miles. Plate II shows the area of the watershed, and the division of this area into the various subdivisions, with the surfaces of these subdivisions. The territory contains some thirteen



PROFILE OF NEW AQU



STATE OF NE

THE AQUEDUCT COMMISSIONERS

MAP OF THE ROUTE

— OF THE —

NEW CROTON AQUEDUCT,

OLD AQUEDUCT

— AND —

BRONX RIVER PIPE LINE

—ALSO THE—

— WATERSHEDS —

OF THE

CROTON, BRONX AND BYRAM RIVERS.

SCALE

1907

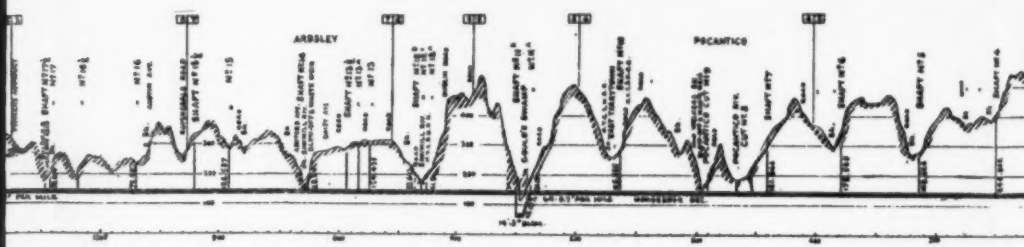
Walter H. Sears

CHIEF ENGINEER

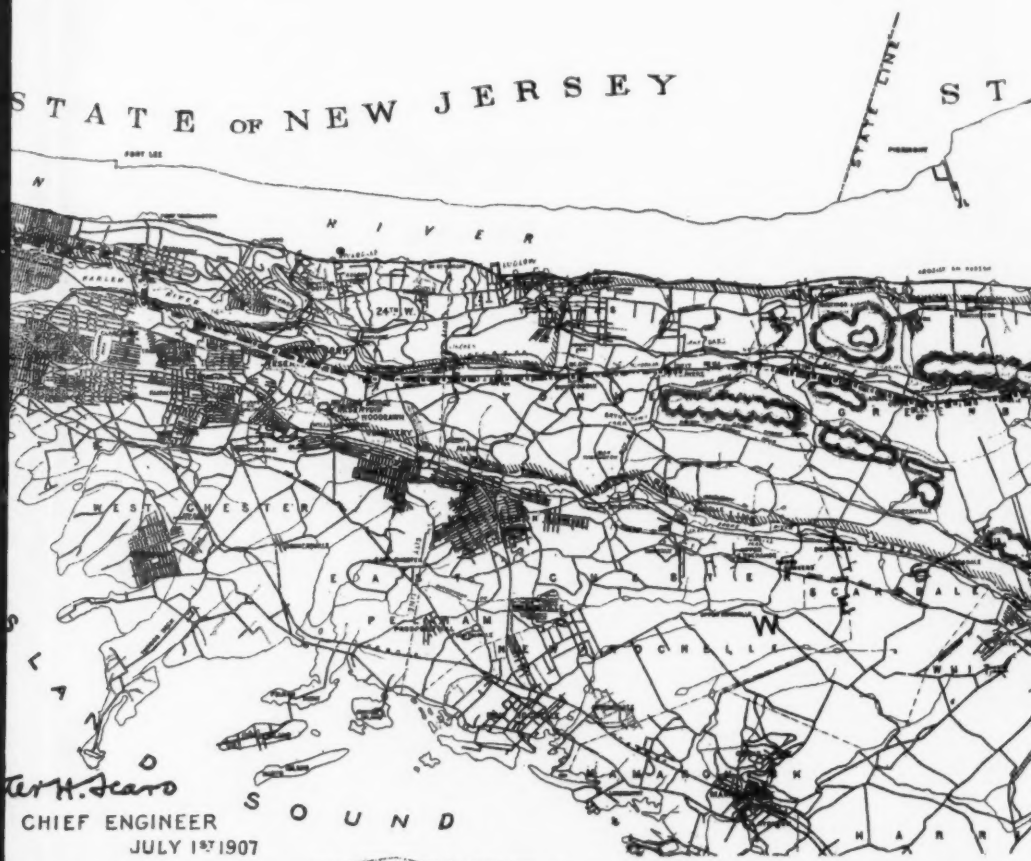
JULY 1st 1907

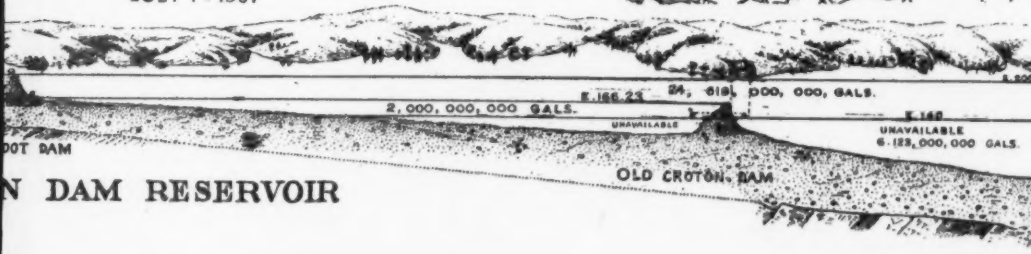
NEW CROTON DAM RESERVOIR

GEORGE A. TABER
Assistant Engineer in Charge
MAIN OFFICE



PROFILE OF NEW AQUEDUCT

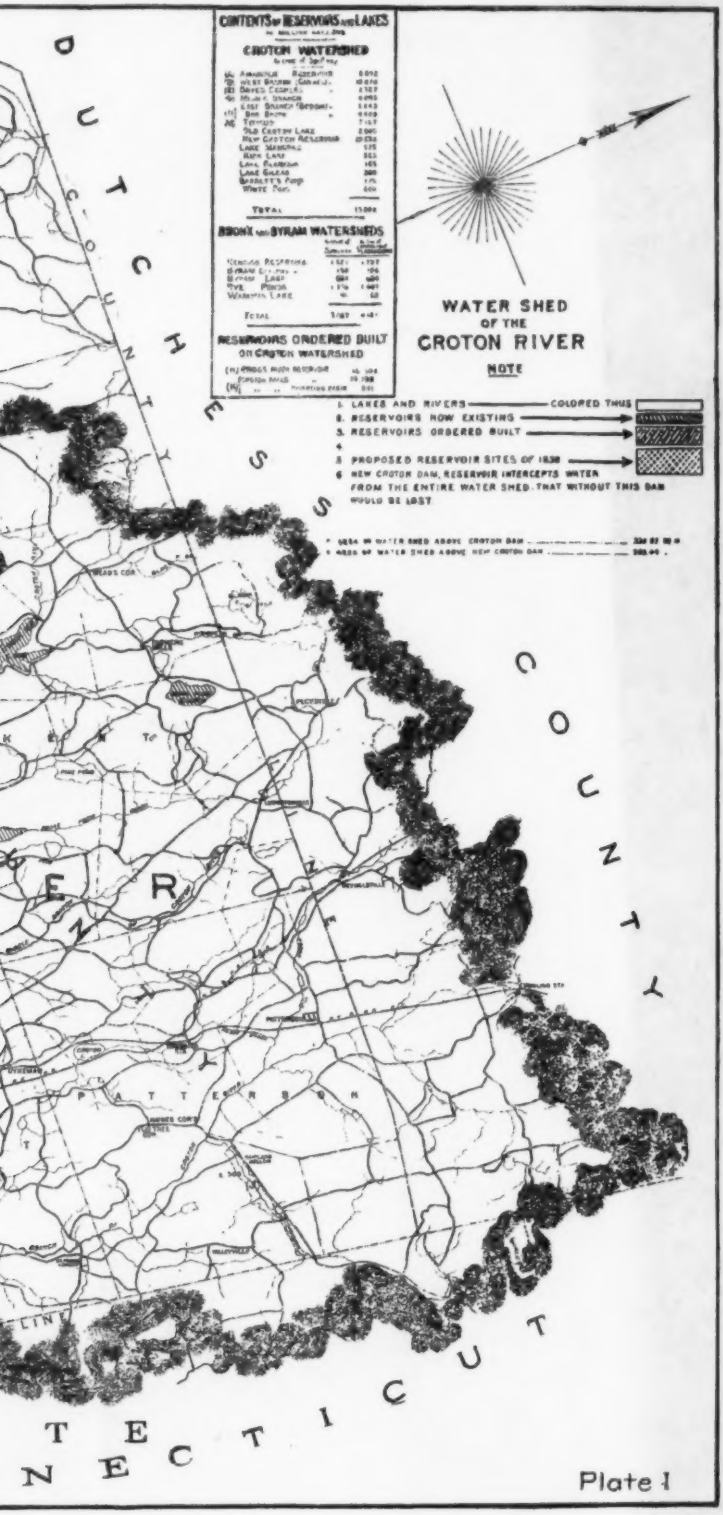




N DAM RESERVOIR

PLATE I. WATER SHED OF THE CROTON RIVER

RIVER
WEST POINT
PUTNAM VALLEY
SHED
STATE
OF
STA
CONN


CONTENTS OF RESERVOIRS AND LAKES
CROTON WATERSHED
BRONX AND BYRAM WATERSHEDS
RESERVOIRS ORDERED BUILT ON CROTON WATERSHED
WATER SHED OF THE CROTON RIVER
NOTE
1. LAKES AND RIVERS — COLORED THUS
2. RESERVOIRS NOW EXISTING
3. RESERVOIRS ORDERED BUILT
5. PROPOSED RESERVOIR SITES OF 1838
6. NEW CROTON DAM, RESERVOIR INTERCEPTS WATER FROM THE ENTIRE WATER SHED. THAT WITHOUT THIS DAM WOULD BE LOST
PUTNAM COUNTY
PATTERSON
CONNECTICUT
Plate I

CITY OF NEW YORK

THE AQUEDUCT COMMISS

CROTON WATER

AREAS OF SUBDIVISIONS (IN SQU

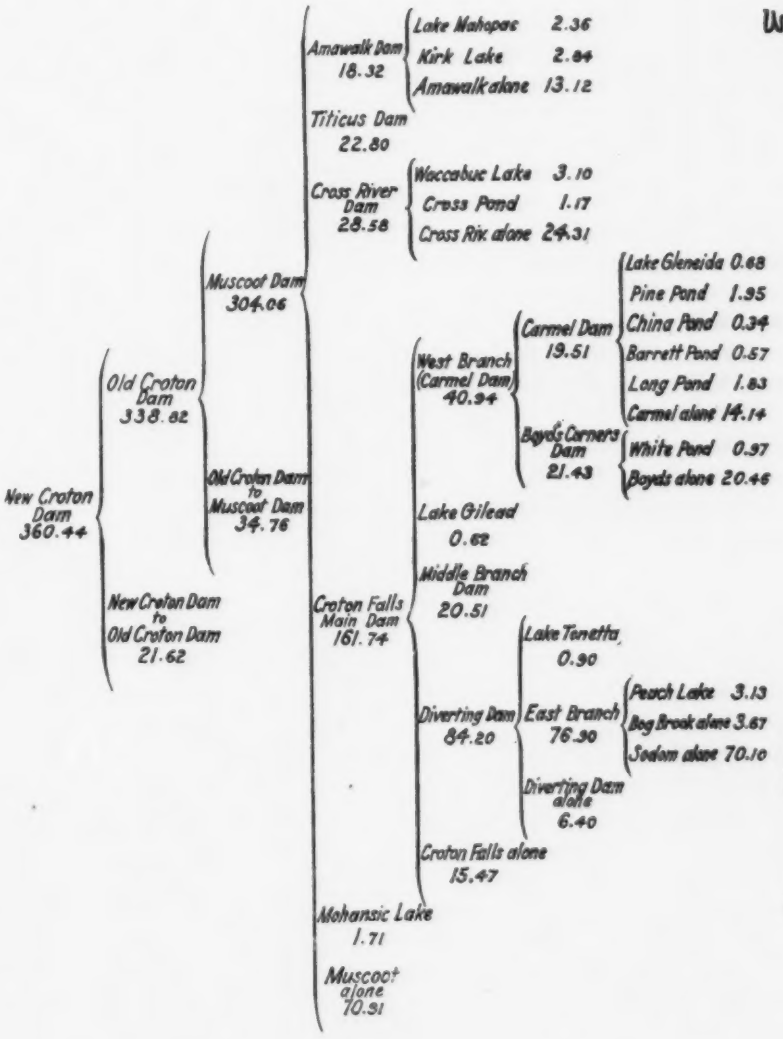


PLATE II. CROTON WATER SHED, AREAS

CITY OF NEW YORK

THE AQUEDUCT COMMISSIONERS

CROTON WATERSHED

AREAS OF SUBDIVISIONS (IN SQUARE MILES)

Walter H. Searo

CHIEF ENGINEER

- Lake Mahopac 2.36
- Kirk Lake 2.64
- Amawalk alone 13.12
- Waccabuc Lake 3.10
- Cross Pond 1.17
- Cross Rix alone 24.31
- West Branch (Carmel Dam) 40.94
  - Carmel Dam 19.51
    - Lake Gleneida 0.68
    - Pine Pond 1.95
    - China Pond 0.34
    - Barrett Pond 0.57
    - Long Pond 1.83
    - Carmel alone 14.14
  - Boyds Corners Dam 21.43
    - White Pond 0.97
    - Boyds alone 20.46
- Lake Gilead 0.62
- Middle Branch Dam 20.51
- Diverting Dam 84.20
  - Lake Tonetta 0.90
  - East Branch 76.90
    - Peach Lake 3.13
    - Bog Brook alone 3.67
    - Sodom alone 70.10
  - Diverting Dam alone 6.40
- Croton Falls alone 15.47

PLATE II. CROTON WATER SHED, AREAS OF SUB-DIVISIONS

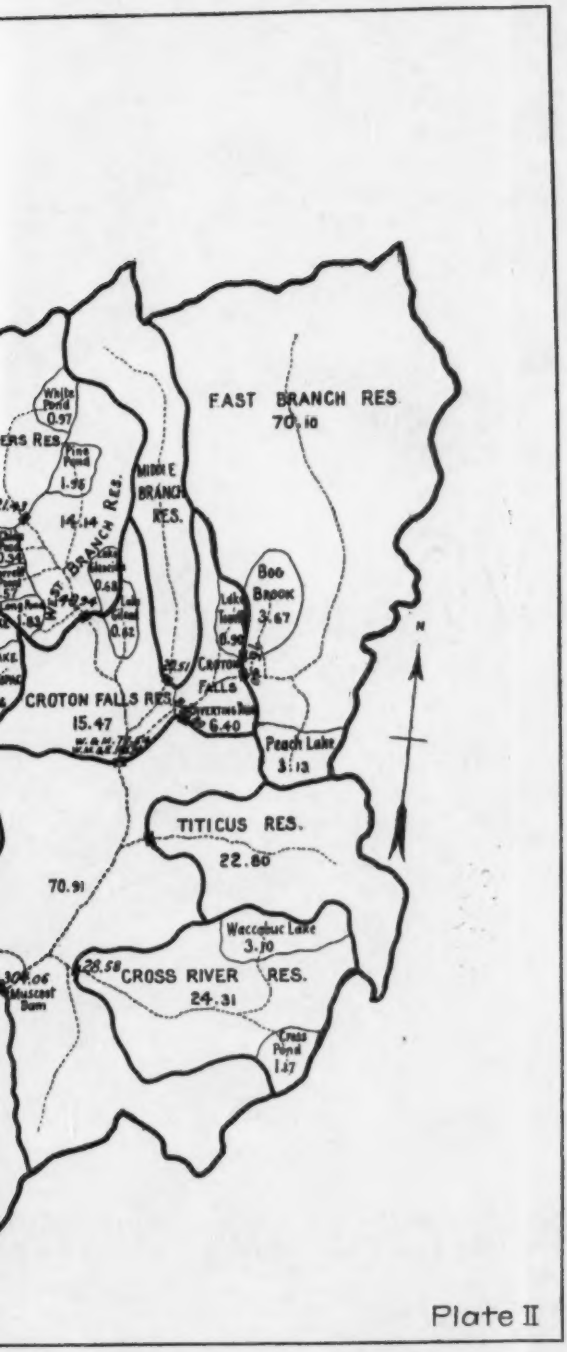

EAST BRANCH RES.
70.10
MIDDLE BRANCH RES.
Boo Brook
3.67
CROTON FALLS RES.
15.47
Croton Falls
Peach Lake
3.13
TITICUS RES.
22.80
70.91
Waccabuc Lake
3.10
CROSS RIVER RES.
24.31
Cross Pond
1.47
N

Plate II

dams, of which two are submerged at high-water level, and a number of natural lakes and ponds. The total capacity to the high-water line of the system in 99,280,000,000 gallons, or to the top of flash boards 104,443,000,000 gallons. Table 1 gives details of these various artificial and natural lakes. All elevations are based on U. S. Coast and Geodetic Survey datum. The water surface is 19.32 square miles or 5.36 per cent of the total area of the watershed.

The amount of water which can ultimately be withdrawn from the reservoirs will be somewhat less than the total available capacity of 104,443,000,000 gallons in the absence of filtration because of the objectionable quality of the water yielded at the end of the draft.

The average rainfall on the Croton watershed over the areas used from 1868 to 1912, inclusive, has been 47.72 inches. The driest year was 1880, with a rainfall of 36.92 inches; the wettest was 1901, with a rainfall of 63.75 inches. The percentage of runoff for these 45 years has averaged 47.71, with a minimum in 1883 of 31.2 per cent and a maximum in 1868 of 66.2 per cent.

Plate III shows for the last ten years the flow of the Croton River at the new Croton dam, and the storage of water in the reservoir system. The net flow of the river varies from 0 to 8.5 second feet per square mile of drainage area, based on monthly averages. A single dry year will seriously affect the available storage. The lowest storage in recent years with the present system of reservoirs was in September, 1911, with 33,030,000,000 gallons available.

CONNECTIONS TO CITY

The water is led to the city through two masonry aqueducts; the first, the old Croton aqueduct, a cut and cover brick horseshoe section, with an area of 53.34 square feet and a usual grade of 0.00021, or 1.11 feet per mile, crossing the Harlem River through an iron pipe line at High Bridge and running to Central Park reservoir; the second, the new Croton aqueduct, a grade tunnel of horseshoe section, with an area of 155.6 square feet and a usual grade of 0.000132, or 0.70 of a foot per mile, to a point about one mile above Jerome Park, where it dips down to a circular pressure tunnel leading to the 135th Street gate house.

Both of the aqueducts lead to the Jerome Park reservoir site, as shown on Plate I. This reservoir has now one completed basin in use with a storage capacity of about 770,000,000 gallons. An area of

TABLE 1

Croton water storage

NAME OF RESERVOIR	ELEVATION, FLOW LINE	AREA	AVAILABLE CAPACITY	HEIGHT, FLASHBOARDS	ADDITIONAL CAPACITY	TOTAL STORAGE	AREA WATERSHED
		sq. miles	*million gals.*	*feet*	*million gals.*	*million gals.*	*sq. miles*
New Croton — Muscoot Dam	199.06	5.574	4,914		2,282	5,706	360.44
New Croton — New Croton Dam	199.06		24,619	2.0		26,110	
Old Croton	165.29	0.760	2,000			2,000	
"A" Amawalk	399.06	0.940	6,692	2.0	394	7,086	18.32
"D" West Branch (Carmel)	502.06	1.560	10,070	1.67	598	10,668	40.94
"E" Boyd's Corner	592.06	0.436	2,727			2,727	21.43
"G" Middle Branch	371.06	0.672	4,005	1.0	150	4,155	20.51
"I" East Branch — Main Dam	416.06	.898	5,243			5,243	73.23
"I" East Branch — Bog Brook	416.06	.640	4,400			4,400	3.67
"M" Titicus	324.06	1.103	7,167	2.0	450	7,617	22.80
"N" Cross River	329.06	1.310	10,308	2.0	615	10,923	28.58
"K" Croton Falls — Main	309.06	1.666	14,192	2.0	673	14,865	161.74
"K" Croton Falls — Diverting	309.06	.244	888			888	84.20
Lake Mahopac	659.06	.875	575			575	2.36
Kirk Lake	582.06	.158	565			565	2.84
Lake Gilead	496.06	.190	380			380	.62
Lake Gleneida	504.06	.284	165			165	.68
Barrett Pond	778.06	.108	170			170	.57
White Pond	829.56	.176	200			200	.97
Total (including small ponds)		19.32	99,280			104,443	360.44

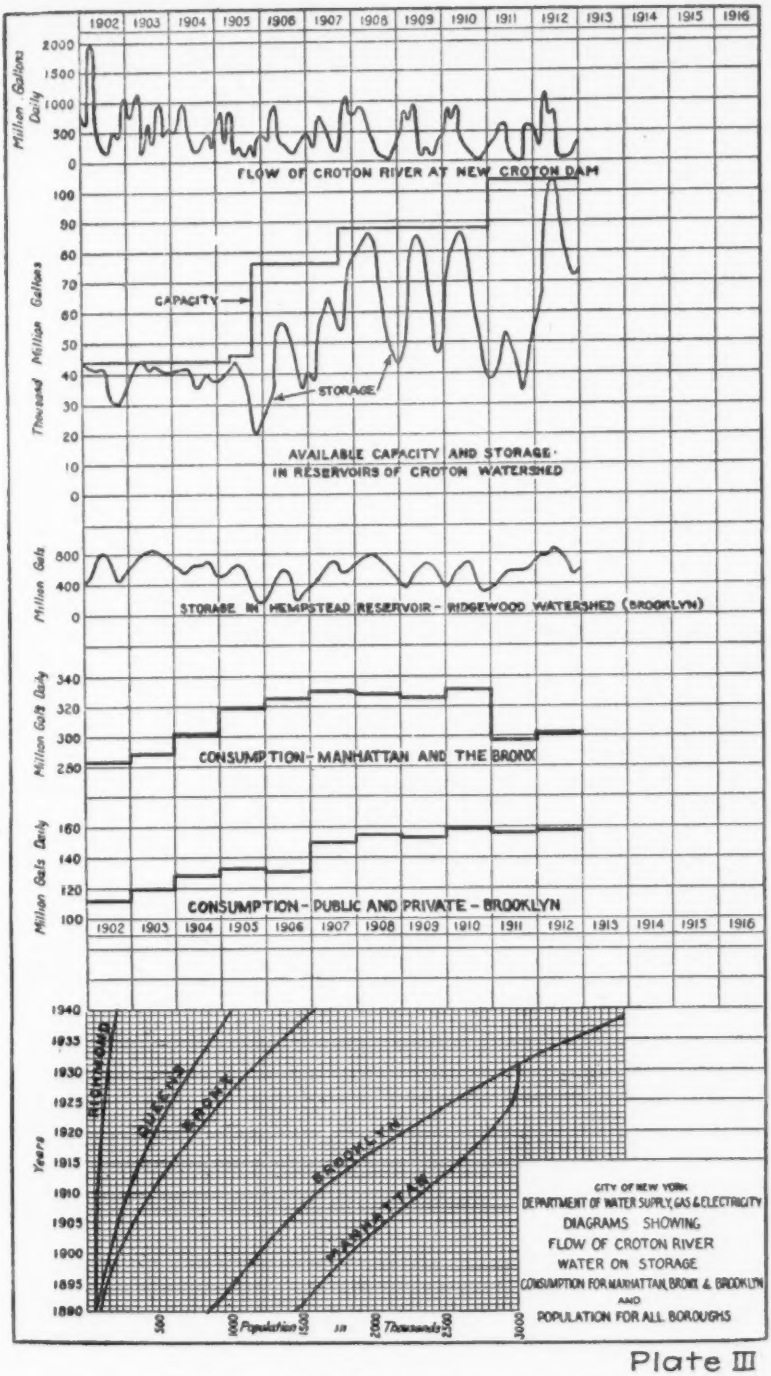


PLATE III. WATER STORAGE AND CONSUMPTION

some 158 acres at the east side of this basin, originally proposed to form a second basin, is at present lying unused, with the excavation largely completed.

QUALITY OF CROTON WATER

The Croton water on average is of fair quality. The turbidity is usually moderate, averaging from 3 to 5 in the city; at times of heavy rains in the watershed, particularly when the level of the reservoirs is low, the turbidity may reach as high as 100.

The color is frequently high, running from 10 to 65 on the platinum scale. The watershed contains much swampy land, and the color is of vegetable origin. Odor is perceptible, and is sometimes very marked. Microscopic growths give the water a distinct taste, sometimes very disagreeable. Bacteria are moderate in number, particularly in the city, after the water is treated with hypochlorite of lime. 1912 records show a range of bacteria at 37° C. of from 5 to 1200 per cubic centimeter. B. coli, in 1912, are found in 1cc in 20 per cent of the tests.

The average alkalinity is 32.

Plates IV and V show graphically the quality of the water for the years 1911 and 1912.

Table 2 gives the average of the analyses for the year 1912. Further data, showing in detail the color and microscopic organisms for the years 1907–1912 and the turbidity for the first part of the year 1912, are given in Tables 3, 4 and 5.

On Plates VI, VII and VIII are shown graphically some data of the characteristics of the Croton water. These curves have been very kindly furnished by Dr. Frank Hale, Director of the Mount Prospect Laboratories.

Plate VI shows the conditions on August 22, 1912. The thermocline is indicated at a depth of 13 feet. Above that line the temperature is high, 23° C., the dissolved oxygen high, 10 parts per million, the carbonic acid low, 1 part per million, and the microscopic organisms high, 6000 per cubic centimeter.

At a deeper level, where the water is quiescent and unaffected by surface influences, the temperature is somewhat lower, 6° C., the dissolved carbonic acid resulting from decomposing vegetation is high, 12.5 parts per million, the dissolved oxygen is very low, 1 part per million, but is always appreciable. The content of microscopic organisms is markedly lower, 800 per cubic centimeter.

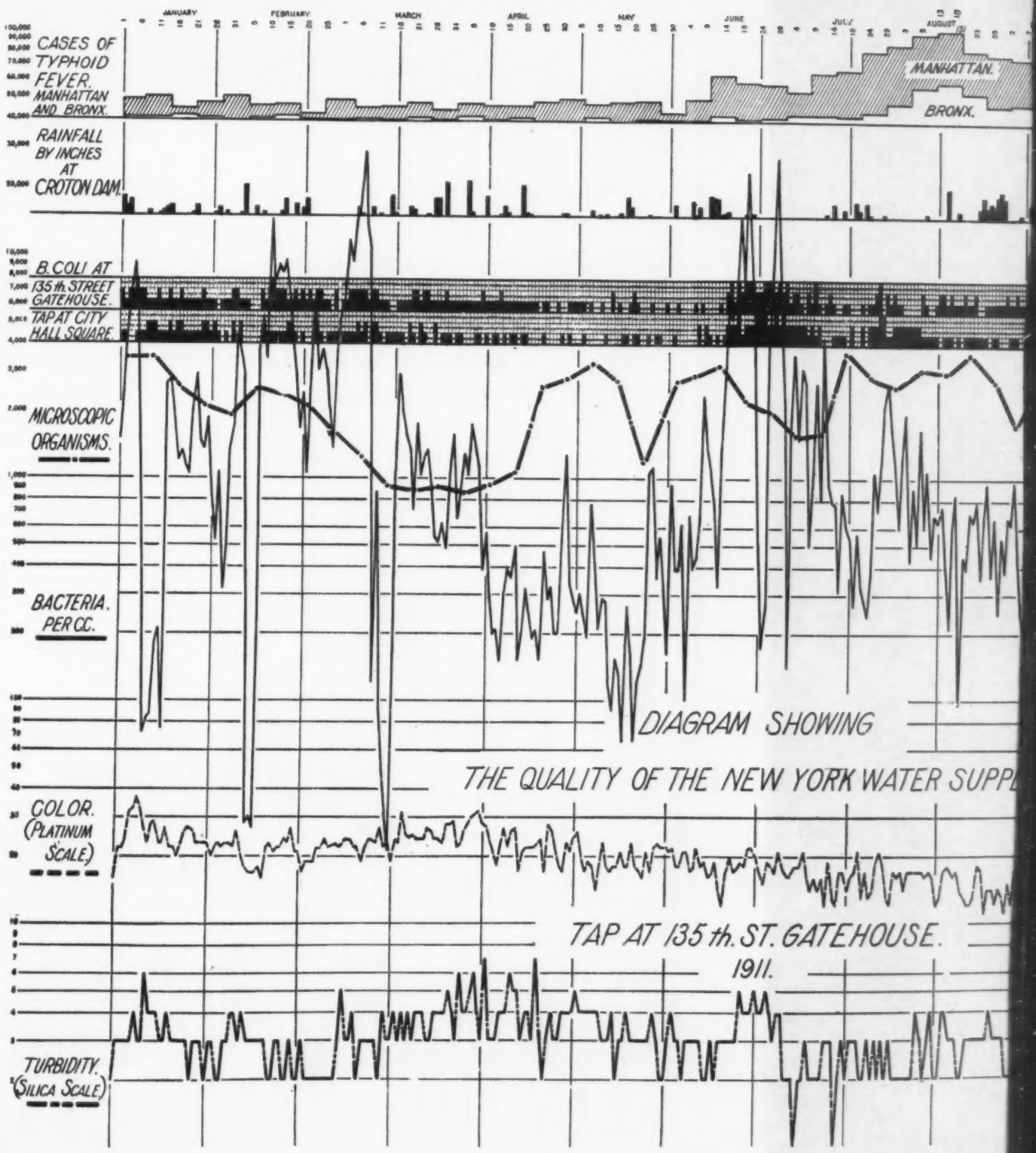


PLATE IV. QUALITY OF CROTON WATER, 1911

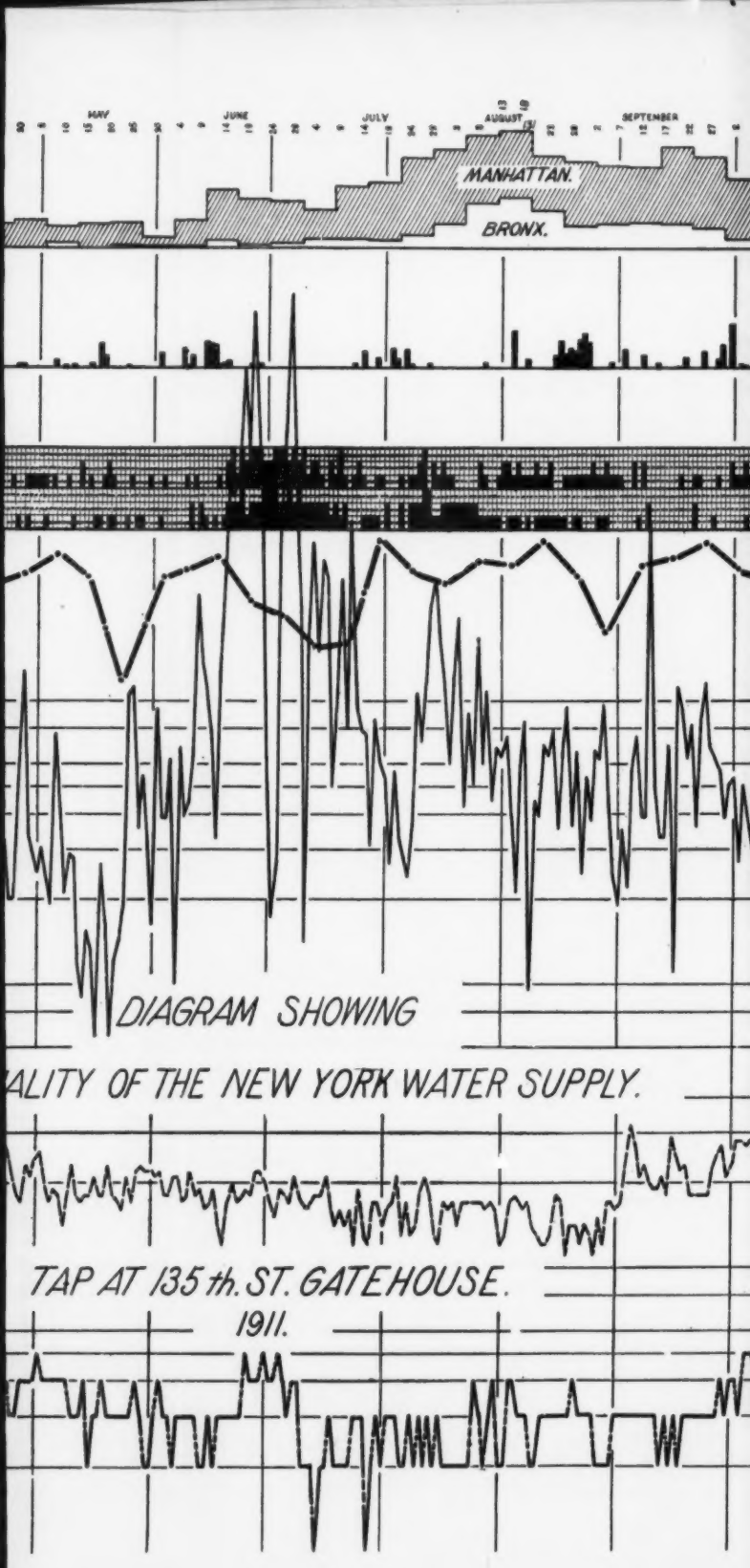


PLATE IV. QUALITY OF CROTON WATER, 1911

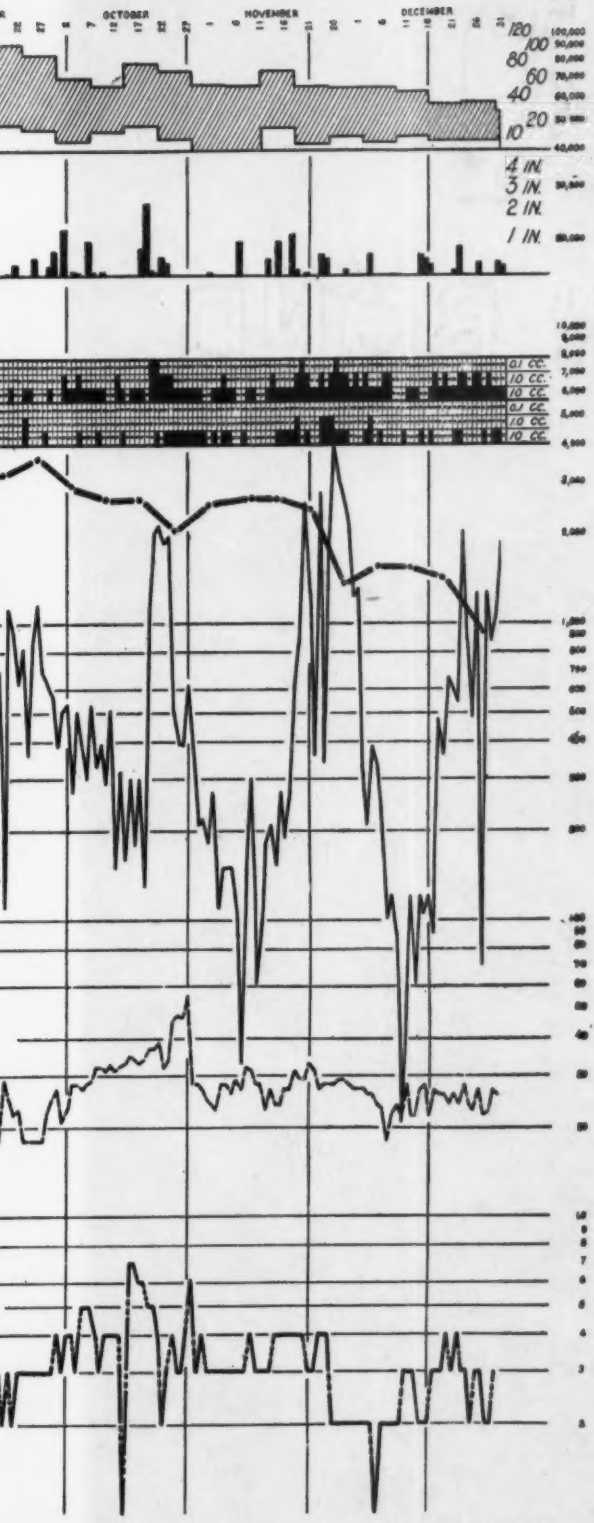

OCTOBER
NOVEMBER
DECEMBER
120
100
80
60
40
20
10
4 IN.
3 IN.
2 IN.
1 IN.
.01 CC.
.10 CC.
1.0 CC.
.01 CC.
.10 CC.
1.0 CC.

Plate IV

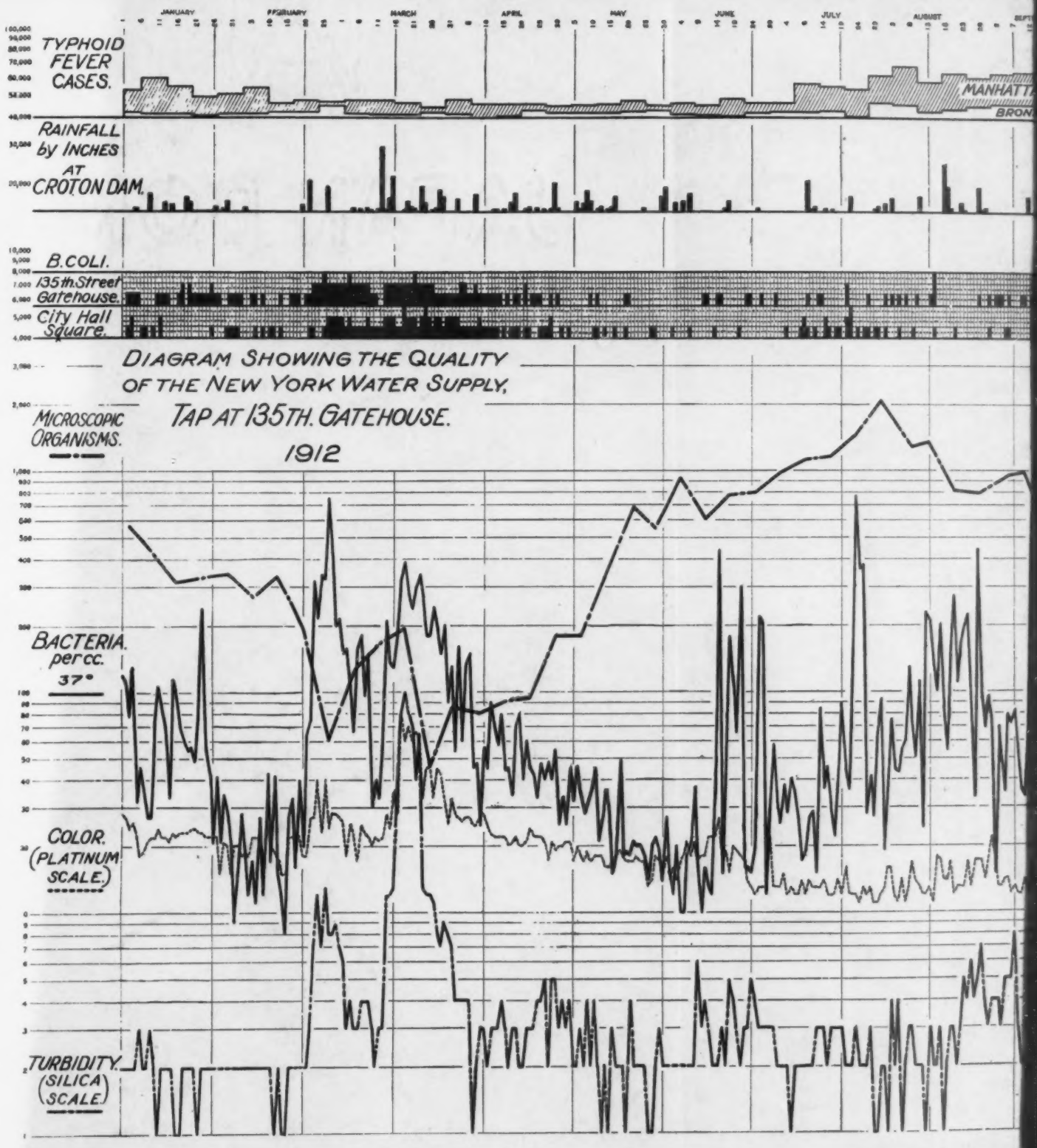
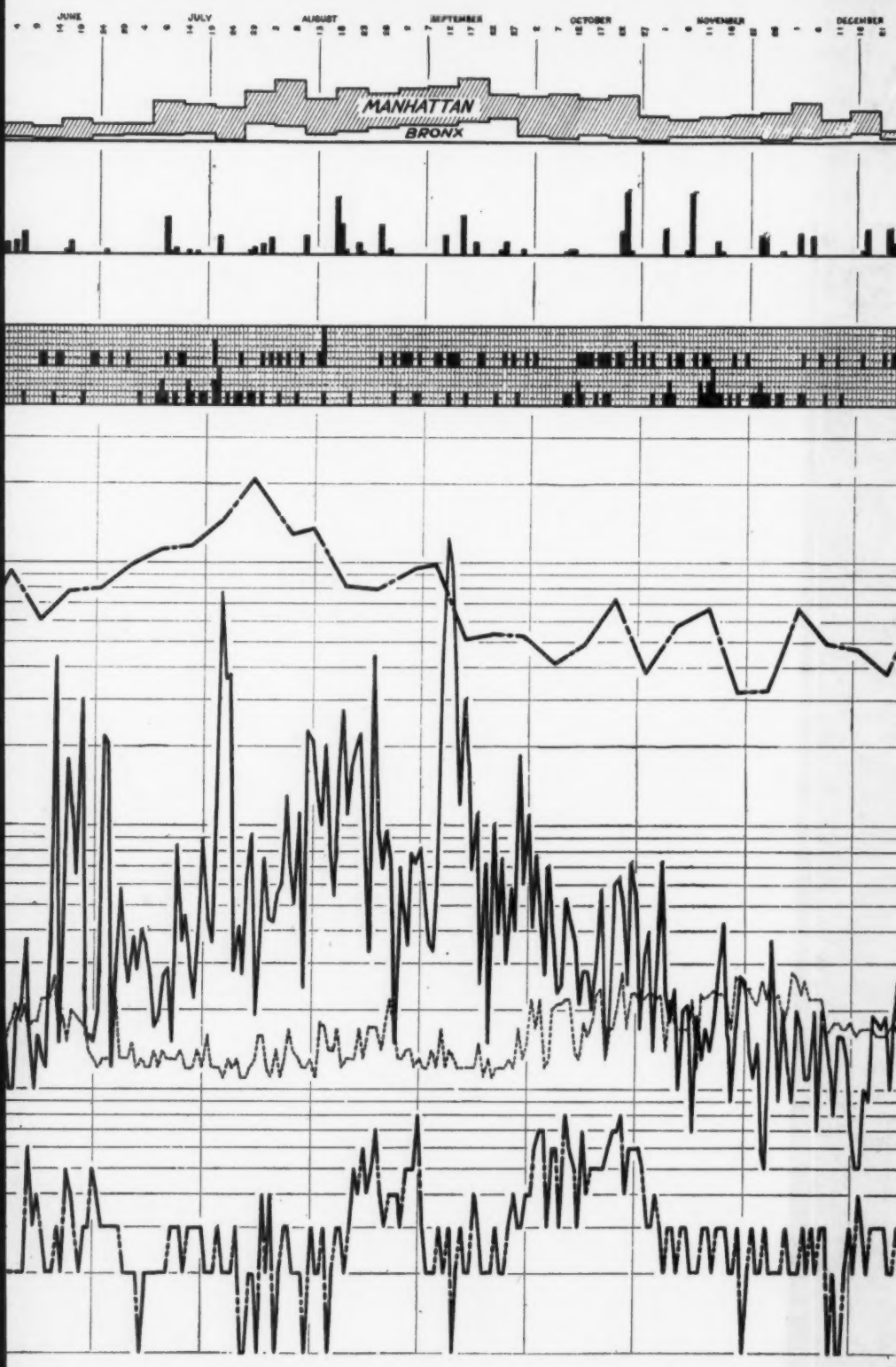


PLATE V. QUALITY OF CROTON WATER, 1912



QUALITY OF CROTON WATER, 1912

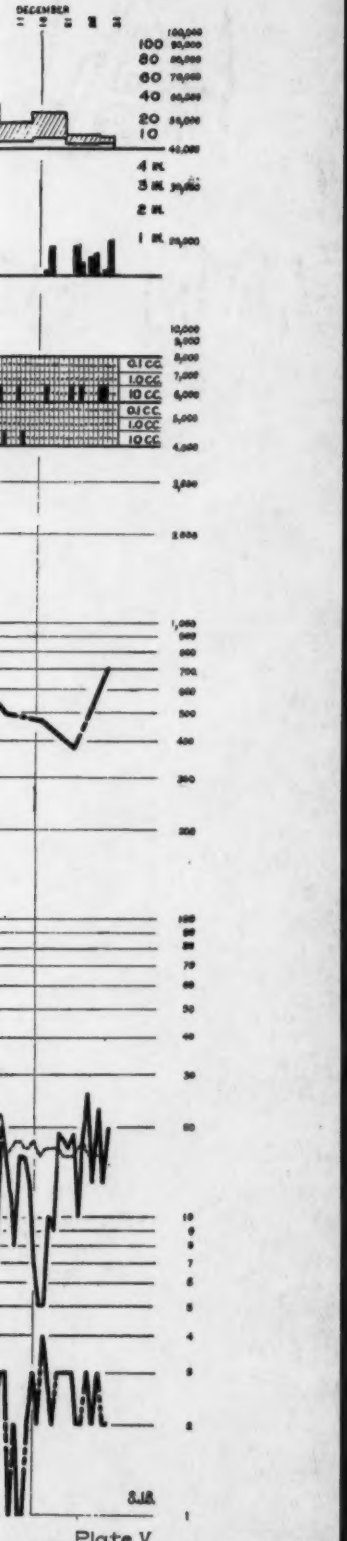

DECEMBER
100
80
60
40
20
10
4 M.
3 M.
2 M.
1 M.
0.1 CC.
1.0 CC.
10 CC.
0.1 CC.
1.0 CC.
10 CC.
S.I.R.

Plate V

The indications from this diagram are that the quality of the water at low levels, as to odor and taste, which result largely from the microscopic organisms, will be materially better. The water is now being drawn in summer time from these low levels with decided improvement in the quality.

Plate VII shows the conditions on October 3, 1912. The fall overturn is under way, and the water is becoming of uniform quality. Further data, showing in detail the color and microscopic organisms for the years 1907–1912 and the turbidity for the first part of the

TABLE 2

Average quality Croton water 1912

KIND OF EXAMINATION	CROTON DAM SURFACE	JEROME PARK	135TH STREET GATE HOUSE	CENTRAL PARK	CITY HALL TAP
Physical:					
Turbidity	10	5	4	4	3
Color	31	19	20	18	18
Odor	12	22	22	22	22
Chemical:					
Albumino'd ammonia:					
In solution	0.153	0.124	0.131	0.152	.....
In suspension	0.062	0.037	0.027	0.021	.....
Total	0.215	0.161	0.158	0.173	.130
Free ammonia	.050	0.080	.059	.046	.028
Nitrites	.003	.003	.005	.005	.003
Nitrates	.08	.19	.209	.19	.21
Total solids	74	75	75	77	.....
Loss on ignition	24	28	25	28	.....
Fixed solids	50	47	50	49	.....
Chlorine	2.5	3.2	3.1	3.2	.....
Hardness	40	39	38	38	.....
Alkalinity	32	32	32	32	.....
Iron	.15	.33	.51	.29	.....
Microscopical:					
Organisms	1,480	848	564	960	496
Amorphous matter	1,710	758	758	592	732
Bacteriological:					
Bacteria per cc. at 37 C.	315	64	79	62	48
Per cent. positive tests for B. coli:					
In 0.10 cc.	19	3	1	2	1
In 1. cc.	35	11	10	20	10
In 10. cc.	69	36	44	.....	41

TABLE 3

Color (parts per million of platinum) Croton water, Jerome Park Reservoir Gate House No. 5

	1907	1908	1909	1910	1911	1912
January	16	27	12	17	22	26
	12	25	13	21	14	23
	30	26	14	16	16	21
	18	23	14	14	22	22
	21	.....	.....	6	17	15
February	17	23	18	19	16	18
	22	29	17	19	16	20
	21	26	13	18	13	18
	.....	23	17	13	16	22
March	21	23	15	16	18	25
	20	22	13	21	13	22
	19	24	14	20	16	65
	14	24	12	17	14	48
	16	29	11	.....	.....	.....
April	14	16	12	23	13	28
	15	16	11	21	14	22
	13	17	8	19	14	20
	13	15	11	18	18	22
	13	.....	.....	.....	.....	18
May	12	15	11	17	10	15
	12	15	12	18	10	.....
	.....	12	12	14	11	.....
	.....	12	14	14	11	10
	.....	.....	.....	11	10	.....
June	14	12	11	13	11	.....
	12	10	12	14	16	.....
	15	12	13	12	16	.....
	11	8	11	12	15	.....
	.....	10	11	.....	.....	.....
July	18	5	12	11	11	.....
	11	10	11	8	18	.....
	12	11	15	11	14	.....
	11	10	11	11	13	.....
	18	.....	.....	.....	14	.....
August	12	10	11	11	11	.....
	13	10	9	11	16	.....
	13	11	13	12	14	.....
	18	12	12	13	14	.....
	.....	12	12	11	.....	.....
September	15	12	16	12	16	.....
	12	17	17	14	22	.....
	12	16	19	14	22	.....
	13	16	16	14	18	.....
	13	.....	.....	.....	.....	.....

TABLE 3—Continued

	1907	1908	1909	1910	1911	1912
October	22	13	13	23	18	.....
	18	13	17	17	23	.....
	14	13	17	12	21	.....
	21	11	12	17	26	.....
	.....	.....	.....	13	29	.....
November	21	11	13	11	28	.....
	18	13	13	11	23	.....
	20	12	13	11	23	.....
	22	12	11	11	29	.....
	.....	13	12	.....	.....	.....
December	22	11	12	13	29	.....
	25	13	15	16	26	.....
	26	13	11	11	28	.....
	26	14	17	12	24	.....
	26	.....	.....	.....	.....	.....
Average	17	16	13	14	18	.....

year 1912, are given in Tables 3, 4 and 5. The thermocline is indicated at 30 feet near the bottom.

Plate VIII shows the conditions on October 17, 1912. The overturn is fully accomplished, and the water has practically uniform characteristics from top to bottom.

POLLUTION OF WATER

The pollution of the water is of various kinds.

1. The watershed is inhabited by about 25,000 people, whose sewage eventually reaches the reservoirs. Some part of these are provided with sewerage systems; a very small part of the population is provided with sewage disposal works.

To protect the water supply as far as is practicable, the city maintains four hypochlorite application plants in the watershed, located at Mount Kisco, Brewster, Lake Gleneida, and Katonah Brook. In addition, all the water passing through the new Croton aqueduct is treated with hypochlorite at Dunwoodie in South Yonkers.

The city also maintains a force of 89 men whose duty it is to patrol the entire watershed systematically, report on and see to the abatement of all nuisances or sources of infection they may discover, and operate the treatment plants. Of the above force, 28 men are regularly detailed and 61 are used for this service, as occasion demands.

TABLE 4

Total microscopic organisms, Croton water, Jerome Park Reservoir, Gate House No. 5

	1907	1908	1909	1910	1911	1912
January	2125	330	3020	2910	4845	540
	2415	490	2065	3105	6590	485
	845	500	2290	1785	5015	335
	1860	600	2990	1585	4650	360
	490	.....	.....	865	2525	345
February	280	455	2550	1135	3000	240
	550	200	5815	965	3540	330
	660	160	5115	715	4575	290
	.....	245	4730	535	4055	100
March	625	540	4475	1075	4225	155
	765	375	3895	1910	4550	200
	250	445	4705	1785	4430	310
	980	220	3510	3105	4275	55
	345	230	3990	.....	.....	.....
April	295	395	4460	2925	4885	85
	1030	660	2070	2550	3545	125
	800	345	1835	2485	3735	140
	1420	365	1510	4530	3805	135
	2265	.....	.....	.....	.....	240
May	2465	695	1030	2805	1070	245
	2170	1185	1195	3375	1820	.....
	.....	655	340	3410	1745	.....
	.....	530	280	3300	1110	.....
	.....	.....	.....	3080	2310	.....
June	1910	350	550	3565	1345	.....
	3315	500	565	3320	1720	.....
	2385	520	170	2425	1435	.....
	1500	455	550	1895	2465	.....
	.....	200	40	.....	.....	.....
July	1580	380	205	2520	4980	.....
	1050	325	155	1075	2470	.....
	655	245	810	800	.....	.....
	23f	215	2530	260	4475	.....
	430	.....	.....	.....	6090	.....
August	260	195	1695	1100	6280	.....
	940	535	1965	1665	5580	.....
	475	435	865	1735	4960	.....
	650	1140	1950	3650	3755	.....
	.....	790	2135	3825	.....	.....
September	1130	1145	3360	5325	3516	.....
	1530	835	3525	7100	3725	.....
	990	1270	2530	9300	4215	.....
	1360	1650	3750	6050	4630	.....
	480	.....	.....	.....	.....	.....

TABLE 4—Continued

	1907	1908	1909	1910	1911	1912
October	2645	16110	4725	7480	3450	.....
	2220	1050	3290	12225	3525	.....
	1150	1300	2960	12680	3630	.....
	2855	1685	4040	9660	2695	.....
	.....	.....	.....	12060	2825	.....
November	6460	1470	3655	13100	2725	.....
	2985	1295	3425	9828	2640	.....
	2615	450	3155	7800	2985	.....
	1500	1465	4540	7138	2895	.....
	.....	1705	5315	.....	.....	.....
December	1385	1660	4275	5160	2190	.....
	2275	1925	5535	5060	1810	.....
	995	780	4515	6548	1820	.....
	820	1805	.....	5360	1365	.....
	175	.....	.....	.....	.....	.....
Average	1435	750	2756	4244	3444	.....

2. Microscopic vegetable growths discoloring the water and giving it a disagreeable taste and odor result from the swampy lands forming part of the reservoir system.

To reduce these as far as possible, copper sulphate is applied by the reservoir forces.

3. The watershed contains some 13 miles of railroad track within 300 feet of the flow line, and 18 railroad bridges aggregating 1700 feet in length. The sewage from the passenger trains enters the reservoirs practically raw, and may often contain pathogenic bacteria. So far, it has not been practicable to enforce proper sanitary control over the wastes from these railroad trains.

The difficulties of asserting authority and abating nuisance are considerable under existing laws, and make watershed protection harder than it should be.

4. Highway bridges, 19 in number, aggregating 4200 feet in length, discharge their wash into the reservoirs.

5. Wash from roads and runoff from farm lands, and the existence of some 24 square miles of swampy land, cause at times a heavy turbidity, and at all times a marked color, bad odor and taste.

TABLE 5

Turbidity of Croton water at 135th Street Gate House 1912

	JANUARY	FEBRUARY	MARCH	APRIL
1	2	2	7	7
2	2	2	6	4
3	2	2	3	4
4	2	2	4	4
5	2	2	3	4
6	2	2	3	4
7	2	2	4	1
8	2	2	4	2
9	3	2	4	3
10	2	2	3	3
11	1	2	2	2
12	2	1	3	3
13	2	2	3	3
14	2	1	12	3
15	2	1	12	4
16	1	2	13	3
17	1	2	40	2
18	2	2	80	3
19	2	2	100	3
20	2	2	80	2
21	2	2	70	.....
22	1	3	40	.....
23	2	7	65	.....
24	2	12	13	.....
25	2	7	12	.....
26	2	12	12	.....
27	2	8	11	.....
28	2	8	8	.....
29	2	9	7	.....
30	2	.....	9	.....
31	2	.....	8	.....

TYPHOID FEVER IN NEW YORK

Typhoid fever has been moderate in New York particularly of late years. Table 6 gives death rates for New York City and other large American cities for the last thirty years.

POTENTIAL DANGER

The possible danger to the health of the community from infection from the polluted drinking water cannot be evaluated into definite figures. Typhoid statistics are a general indication of the

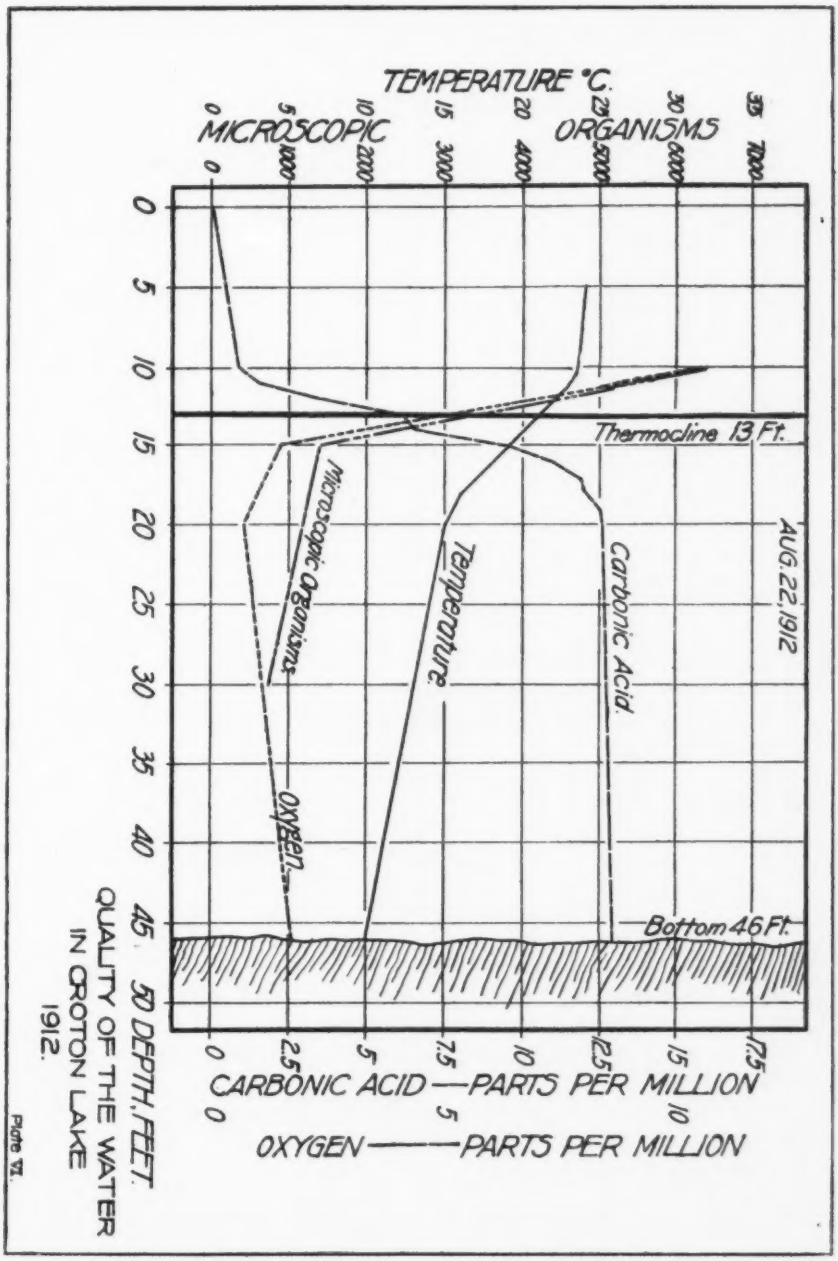

TEMPERATURE °C.
0 5 10 15 20 25 30 35
MICROSCOPIC ORGANISMS
0 1000 2000 3000 4000 5000 6000 7000
Thermocline 13 Ft.
Microscopic Organisms
Temperature
Carbonic Acid.
Oxygen
AUG. 22, 1912
Bottom 46 Ft.
DEPTH, FEET.
0 5 10 15 20 25 30 35 40 45 50
CARBONIC ACID — PARTS PER MILLION
0 2.5 5 7.5 10 12.5 15 17.5
OXYGEN — PARTS PER MILLION
0 5 10
QUALITY OF THE WATER IN CROTON LAKE 1912.
Plate VI.

PLATE VI. QUALITY OF CROTON WATER

TABLE 6

Annual typhoid fever death rates for various American cities. In deaths per 100,000 population

YEAR	CHICAGO	MILWAUKEE	DETROIT	CLEVELAND	BUFFALO	TORONTO	BOSTON	NEW YORK	MANHATTAN AND THE BRONX	PHILADELPHIA	BALTIMORE	WASHINGTON	CINCINNATI	PITTSBURGH	SAN FRANCISCO	ST. LOUIS	NEW ORLEANS
1880	34	37	.....	44	.....	44	42	25	.....	58	59	54	70	135	34	40	24
1881	105	47	.....	99	67	66	56	38	.....	74	58	86	71	151	37	53	30
1882	82	32	.....	68	69	71	57	32	.....	73	47	66	55	158	61	45	33
1883	62	25	.....	65	34	81	52	32	.....	64	35	62	51	107	64	41	23
1884	56	31	.....	60	42	63	56	29	.....	71	42	76	56	70	58	42	25
1885	75	28	.....	34	23	59	40	27	.....	64	42	65	42	76	44	31	17
1886	69	30	39	56	28	46	34	26	.....	64	39	71	54	68	49	30	13
1887	50	31	65	52	34	61	44	26	.....	63	41	78	142	128	23	28	15
1888	47	42	46	47	29	52	40	23	.....	77	42	87	70	87	33	31	19
1889	48	28	31	74	30	46	43	24	.....	71	47	94	49	95	55	33	17
1890	92	41	19	69	40	93	35	22	.....	64	59	112	69	132	44	31	21
1891	174	33	34	50	51	100	34	23	.....	64	35	82	62	101	42	29	24
1892	124	30	94	59	34	47	29	22	.....	40	44	89	40	100	32	93	20
1893	54	35	42	52	37	45	31	21	.....	41	49	86	44	111	32	44	15
1894	38	25	28	29	59	26	29	17	.....	33	47	91	55	56	37	34	29
1895	38	26	24	35	26	27	33	17	.....	40	36	86	39	78	34	21	44
1896	53	18	23	43	20	27	31	16	.....	34	38	58	52	62	26	20	33
1897	29	12	15	23	20	18	33	16	.....	33	37	48	32	64	19	23	52
1898	41	17	22	34	27	16	34	21	.....	52	36	71	33	73	41	17	66
1899	27	17	13	32	24	20	30	16	.....	75	29	73	37	111	22	23	55

1900	20	17	27	54	26	20	26	21	.....	35	35	78	39	144	13	29	40
1901	29	22	21	36	27	17	25	20	.....	34	27	60	55	125	22	34	48
1902	45	15	24	33	33	13	24	21	.....	44	42	78	62	134	27	37	45
1903	32	17	20	114	35	16	20	17	.....	69	36	48	42	134	.....	47	39
1904	20	14	20	48	23	23	23	17	.....	53	37	47	79	142	27	36	36
1905	17	23	21	15	23	16	20	16	.....	48	36	48	40	99	24	20	32
1906	19	31	24	20	23	26	20	15	.....	72	35	50	70	130	.....	17	30
1907	18	26	35	19	28	22	10	17	.....	59	41	35	45	125	.....	15	55
1908	16	17	22	13	20	20	25	12	11	35	33	37	18	45	.....	14	33
1909	13	23	23	13	23	23	14	12	12	21	25	33	13	23	.....	16	29
1910	14	45	20	19	20	41	12	12	11	17	42	23	6	28	.....	13	32
1911	11	19	17	15	25	20	9	11	10	14	27	21	11	26	.....	11	31
1912	8	25	18	6	11	12	8	10	7	13	24	22	7	13	14	14	14

NOTE: New York includes Borough of Manhattan only through 1898—since 1898 Greater New York.

average sanitary quality. A high typhoid death rate shows generally a dangerous water; a low rate, such as ours in New York, shows a water of safe average quality. It does not, however, show that the water may not, at some future time, become grossly polluted by bacteria causing diseases such as typhoid or other intestinal diseases, in the watershed, and such infection may be transmitted to the city.

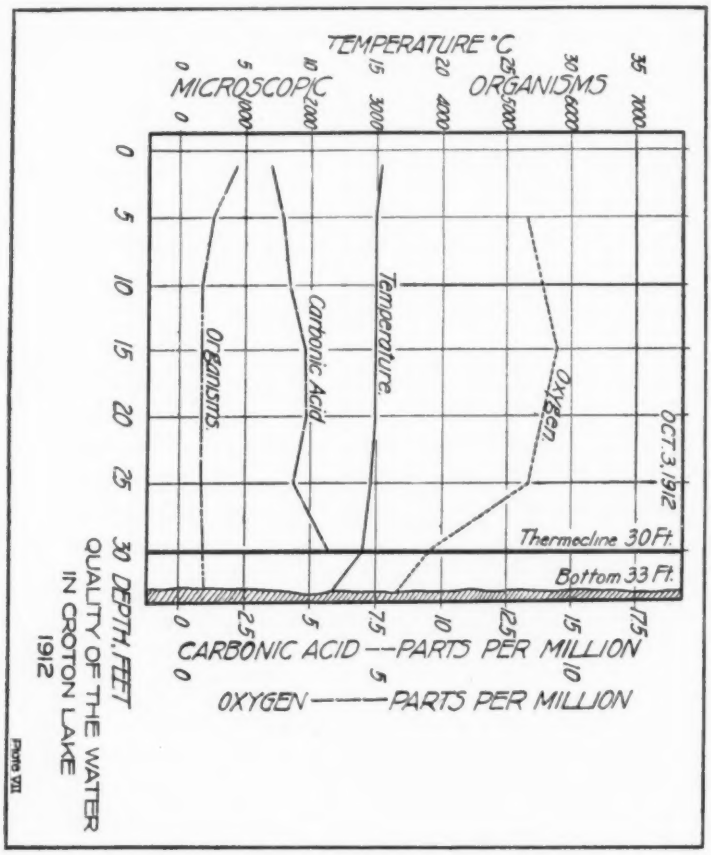


PLATE VII. QUALITY OF CROTON WATER

For some years the water reaching the city has been treated with hypochlorite, and the grosser sources of pollution in the watershed are individually treated. But, while such treatment we know to be fully effectual and reliable when treating a filtered water or a clear water of small organic content, it is only a rough palliation for waters such as the Croton.

PRELIMINARY STEPS TOWARD PURIFICATION

In 1899 the New York City Board of Health recommended the filtration of the Croton water.

Mr. John R. Freeman states in his report of March 23, 1900, on the Water Supply of New York:

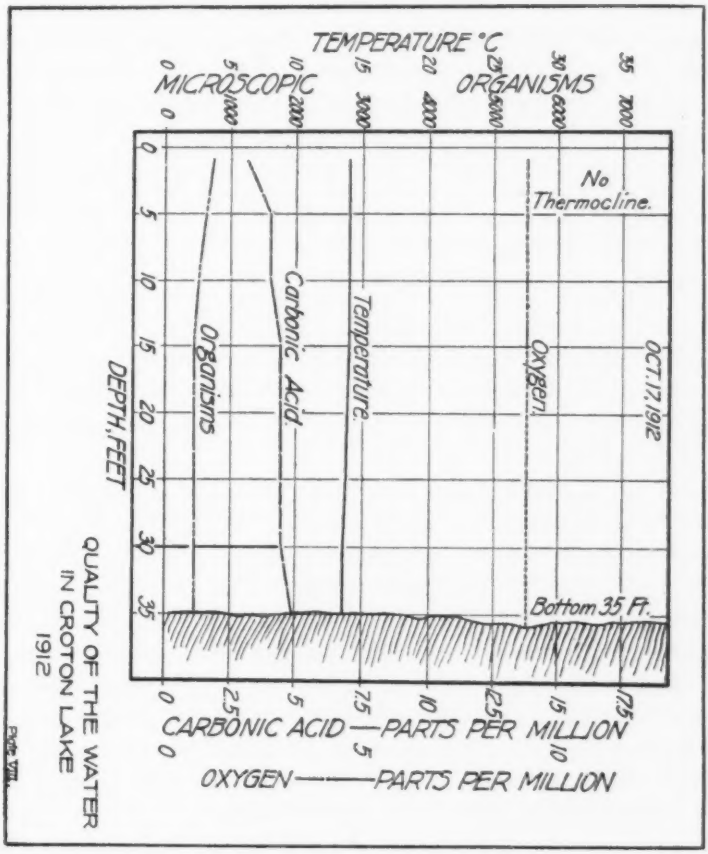


PLATE VIII. QUALITY OF CROTON WATER

I venture to express the belief that by 20 years hence the public will have become educated to demand a higher standard of purity in public water supplies, and all future work should be laid out with a view to filtration 10 or 20 years hence of all water entering the distribution system.

In 1903, the Burr-Hering-Freeman Commission on the Additional Water Supply for the City of New York stated:

> Regarding the Croton water, the Commission urges that suitable lands be secured at once upon which to erect a filter plant sufficient in size to purify the entire supply obtained from this source.

On November 22, 1904, Mr. J. Waldo Smith, then Chief Engineer of the Aqueduct Commission, recommended to the Commission the use of the easterly half of the Jerome Park site for a covered reservoir for the filtered Croton water, and the employment of Messrs. Hazen and Whipple to report on this project. Hazen and Whipple reported favorably on filtration on December 12, 1904.

On July 25, 1905, Dr. Thomas Darlington, Commissioner of Health, reported:

> A careful study of the conditions on the Croton watershed and of the various other supplies of the Greater City, covering a period of many months, has convinced me of the necessity for the establishment of filtration plants.

On January 16, 1905, the Burr-Hering-Freeman Commission, reported on the recommendation of Mr. J. Waldo Smith, recommending the suspension of work on the east basin of the Jerome Park reservoir, in order to permit of the use of this site for a covered reservoir. This recommendation was adopted by the Aqueduct Commission on December 21, 1905.

On March 28, 1906, Mr. I. M. de Varona, Chief Engineer, Department of Water Supply, Gas and Electricity, submitted a report to the Commissioner of the Department on sites for a filter plant and recommended that the Jerome Park site be used for the filters themselves, as well as for the reservoir. On May 23, 1906, the Board of Estimate granted an appropriation for the purpose of investigating the Croton water supply and preparing plans for filter construction.

Messrs. Hering and Fuller were appointed as consulting engineers for this investigation. The results of this investigation were embodied in a report dated October 30, 1907, by Mr. I. M. de Varona, approved by Messrs. Hering and Fuller.

The recommendation was made to build 42 acres of slow sand filters, superposed on a filtered-water reservoir; these filters would be operated at a normal rate of 10,000,000 gallons per acre per day. The filters would be cleaned by the Blaisdell washing machine. Coagulation was to be employed during eight months of the year; a settling basin of eighteen hours' capacity was to be provided, and a clear-water reservoir of 575,000,000 gallons capacity. To bring the

water to the filters, the water level at the reservoir would be raised 1½ ft.; the new Croton aqueduct would supply 310,000,000 gallons per day, and the old Croton aqueduct 90,000,000 gallons per day, as a maximum.

This project was pigeonholed, very likely because the panic of that year and the difficulty of selling bonds made public improvements of this character impracticable.

On May 18, 1911, on the recommendation of the Commissioner of Water Supply, Gas and Electricity, an appropriation of $8,690,000 was made by the Board of Estimate and Apportionment for the construction of the Jerome Park filters. The ordinance for the issue of corporate stock was adopted by the Board of Aldermen June 20, 1911, and took effect July 3, 1911.

DESIGN OF FILTER PLANT

Plans for the construction were made under the direction of Mr. I. M. de Varona, Chief Engineer of the Department, the writer as Consulting Engineer, and Mr. E. G. Manahan, Division Engineer, in immediate charge.

COMPARISON OF TYPES OF FILTERS

The first step was a thorough investigation of the relative merits of slow sand filters both with and without coagulant and of mechanical filters. A set of general plans was drawn for each type of construction and comparative estimates made of the costs of construction and operation.

The results of this investigation are embodied in a report by Mr. I. M. de Varona dated May 20, 1912. Tables 7, 8 and 9 given herewith show the costs of construction, operation, and fixed charges, for slow sand filters, modified sand filters and mechanical filters.

The above tables are based upon an average quantity filtered of 320,000,000 gallons daily.

Fixed charges include interest on investment at 4 per cent plus ¾ of 1 per cent for extraordinary depreciation as distinguished from repairs and renewals of parts.

The recommendations resulting from this investigation are stated in a report by the writer dated May 21, 1912, as follows:

TABLE 7

Jerome Park filters. Comparative estimates of sand and mechanical filters

	SAND FILTER PLANT						MECHANICAL FILTER PLANT		
	Without Coagulation. Rate 5,000,000 gallons daily			With Coagulation. Rate 8,000,000 gallons daily			Rate 125,000,000 gallons daily		
	Area	Capacity	Cost	Area	Capacity	Cost	Area	Capacity	Cost
	acres	*million gals.*		*acres*	*million gals.*		*acres*	*million gals.*	
Settling reservoirs, including mixing chamber, head house, laboratory, drainage and inlet conduit for filters			$262,000	34	280	$2,401,600	12	88	$1,690,000
Filters and equipment, including filter-water reservoir beneath	76		7,880,000	48		4,971,100	4.6		2,080,000
Filtered-water reservoir (part not beneath filters)	43		1,565,400	43		1,565,400	55		2,216,000
Filtered-water reservoir:									
Total capacity		640			490			365	
Capacity above El. 123		56			42			54	
Gate house changes and additions			233,000			233,000			283,000
Changes in aqueducts and west basin			60,000			60,000			60,000
Plant for power and heat			120,000			120,000			110,000
Engineering and contingencies			2,024,100			1,870,200			1,288,000
Total			$12,144,500			$11,221,300			$7,727,000
Less cost of supervision (to come from Budget, and estimated at 5 per cent)			506,000			467,600			322,000
Total from Appropriation			$11,638,500			$10,753,700			$7,405,000

TABLE 8

Jerome Park filters. Comparative charges for a sand filter plant and a mechanical filter plant per million gallons filtered (based on an average of 320,000,000 gallons filtered daily)

	SAND FILTERS		MECHANICAL FILTERS
	Without coagulation	With coagulation	
Administration and laboratory.....	0.23	0.23	0.23
General work and caring for grounds	0.11	0.12	0.13
Operating and cleaning filters.....	0.25	0.25	0.11
Chemicals and their application.....	0.08	1.33	1.73
Light, heat and power.....	0.64	0.64	0.66
Repairs, renewals and supplies.....	0.28	0.30	0.25
Cleaning, settling basins and disposal of sludge.....	0.00	0.10	0.10
Cost of Operation.....	$1.59	$2.97	$3.21
Interest on cost of construction; and extraordinary depreciation as distinguished from repairs and renewals of parts; 4¾ per cent of cost.....	4.94	4.56	3.14
	$6.53	$7.53	$6.35

TABLE 9

Jerome Park filters. Comparison of sand and mechanical filters. Showing investment cost, cost per million gallons filtered, and annual cost

	TOTAL INVESTMENT	FIXED CHARGES PER MILLION GALLONS	COST OF OPERATION PER MILLION GALLONS FILTERED	TOTAL COST PER MILLION GALLONS FILTERED	TOTAL ANNUAL COST
Sand filter plant without coagulation rate 5 m.g.d.....	$12,144,500	$4.94	$1.59	$6.53	$763 000
Sand filter plant, with coagulation; rate 8 m.g.d.....	11,221,300	4.56	2.97	7.53	880,000
Mechanical filter plant	7,727,000	3.14	3.21	6.35	742,000

CONCLUSIONS AND RECOMMENDATIONS

Plain sand filtration will not remove uniformly the turbidity and color in the Croton water sufficiently to obtain a clear, colorless water at all times. In fact, there would be a noticeable vegetable stain in the filtered water obtained with this type of filter for some six or eight months during the average year. Following very heavy rains the effluent of plain sand filters would also show somewhat excessive turbidities due to silt and clay. Such conditions prevailed during the latter part of the past winter.

With well managed plants of either of the three types satisfactory results would be uniformly obtained as to freedom from objectionable bacteria and obnoxious odors.

While a plant of the plain sand filter type would not uniformly produce a water of satisfactory appearance as to color and turbidity, it is perfectly feasible to obtain at all times a fully satisfactory grade of filtered water by either the modified sand filter method or by mechanical filters. So far as quality of filtered water is concerned, there is no choice between these methods. Each would be fully satisfactory as to efficiency.

The problem, then, becomes one of considering whether mechanical filters or modified sand filters would produce the desired result at least cost, when consideration is given both to the first investment and to the cost of operation and maintenance. On the latter score the estimates which have been prepared leave no room whatever for doubt. Mechanical filters, when compared with modified sand filters, show a saving of first cost of practically $3,500,000 and an annual saving of $138,000 when fixed charges are added to operation and maintenance of each type of filter.

I am firmly convinced, therefore, of the soundness of the conclusion that it is best to build mechanical filters for the purification of the Croton water supply and herewith I formally and unqualifiedly recommend the adoption of filters of the mechanical type.

The plans and estimates so made, with the data on which they were based, were then submitted to a consulting board of water purification experts, appointed by the Commissioner of the Department; this board consisted of Messrs. Allen Hazen, George A. Johnson, Frank A. Barbour, W. P. Mason and James H. Fuertes.

The points submitted to this board of experts for investigation were the following:

Mechanical filters

1. Are the mechanical filters, as shown by the plans of the Department, in accordance with modern practice?

2. Will these mechanical filters properly purify the water and render it clear and colorless?

3. Are the estimates of cost of constructing these mechanical filters reasonable?

4. Is the estimated cost of operation of these mechanical filters reasonable?

Sand filtration

1. Will it be necessary to use coagulants in order to obtain clear and colorless filtered water at all seasons of the year?

2. Are the sand filters, as shown in the general plans of the Department, in accordance with modern practice where coagulants are to be used?

3. Will these modified sand filters properly purify the water and render it clear and colorless?

4. Are the estimates of cost of constructing these sand filters reasonable?

5. Is the estimated cost of operation of these sand filters reasonable?

The Board reported to the Chief Engineer on June 10, 1912, and the substance of the report may be summarized by quoting from the Chief Engineer's letter dated June 20, 1912, as follows:

I enclose herewith copy of report made by the Consulting Board of Experts on the plans for the filtration of the Croton water supply, answering the questions propounded by us.

They report that "the mechanical filters shown by the plans are in general accordance with modern practice."

That they believe that they will properly purify the water and render it clear and colorless.

That the estimates of cost and operation appear to them to be reasonable.

That, in order to produce clear water, and water that is sufficiently colorless to be satisfactory at all times, with sand filters, it will be necessary to use coagulants for a portion of the time.

That "it is open to doubt whether the Jerome Park site can be considered at the present time as one permitting of suitable plans for sand filters of such type" as they should be willing to call generally in accord with modern practice.

These conclusions fully establish those of the preliminary report made by me to you, in which the construction of mechanical filters was advocated.

The suggestions made by the Board as to aeration, use of sulphate of copper, etc., are in accordance with the views of this Bureau and will receive due and timely consideration.

The detail plans and specifications for the mechanical filter plant had in the meantime been in the course of preparation, and were completed June 25, 1912.

DETAIL PLANS FOR MECHANICAL FILTERS

Site. The general appearance of the site at Jerome Park looking from the west to the east is shown on Plate IX; a map of the site and the surrounding streets is shown on Plate X. A topographical plan of the site is shown on Plate XI.

Hydraulics. The existing west basin, which contains the Croton water, has a flow line at elevation 131.5, Public Works datum. (This datum is 2.67 feet above United States Coast and Goedetic Survey datum, and is 3.61 feet above Croton datum.) A moderate amount of change of the walls and banks of this basin would permit raising the flow line to elevation 133, and this elevation was adopted. A loss of 7 feet through the whole filter plant was allowed and the high elevation for the filtered-water reservoir fixed at 126; a minimum elevation in the filtered-water reservoir of 123 would permit of proper supply to the City at a maximum rate of 400,000,000 gallons per day, with the Central Park reservoir at elevation 111.

The supply through the new Croton aqueduct was taken as a maximum of 310,000,000 gallons per day. A careful study of all avaiable data on the resistance and delivery of this aqueduct, particularly of a series of gagings made by the Board of Water Supply, led to the conclusion that the delivery of the aqueduct when practically full could be represented by the formula

$$V = 134 \sqrt{r.s.}$$

where v is velocity in feet per second,

r the hydraulic mean radius,

s the slope.

The hydraulic gradient of the aqueduct under these conditions, assuming a coefficient in the Chezy formula of 130, is given in Table 10.

At such places where the internal pressure on the aqueduct makes it necessary the aqueduct was to be suitably reinforced by additional earth cover. Minor changes in blowoff houses were required to suit the raw-water elevations. The maximum discharge of the old Croton aqueduct was taken from Mr. John R. Freeman's Report (Fig. 43) as 90,000,000 gallons per day.

The average yield would require practically only the capacity of the new aqueduct. For greater demands, a pumping equipment lifting the water in the old aqueduct over a static head of about $5\frac{1}{2}$ feet was projected.

Of the 7 feet of effective head for the filter plant, 1 foot was required

PLATE IX. SITE OF FILTER PLANT

for the mixing chamber, 3 inches for the Venturi meter, 9 inches for various gate chambers, channels, and for the coagulating basins; the remainder, 5 feet, was available for the filters proper.

AERATION

The needs and possibilities of aeration were studied without arriving at a conclusion. Plates VII and VIII showing deficiencies in dissolved oxygen, and large content of carbon dioxide, together with the growth of microscopic organisms, indicate but do not establish that eration might be helpful. Estimates were made for locating an aeration plant at Pocantico cut, about 10 miles below the new Croton dam, and the installation of the aerating plant would require a raising of the working water level above it of 15 to 20 feet. This would for a good part of the time have been feasible.

A series of practical tests of the water was required before definitely adopting aeration as a necessary part of the water treatment.

THE TURBIDITY AS AFFECTED BY THE CATSKILL SUPPLY

Some study was given to the possible effect of the use of the Catskill supply in times of high turbidity as a substitute for the Croton. The question considered was, if it would be practicable to cut off the Croton supply at such times, using only Catskill water, until the Croton water became less turbid.

The indications are that the Catskill water will at such times be likely to suffer from excess turbidity as well as the Croton water. From all information available it did not seem probable that the Catskill water would be any better at such periods, and it was decided that no reliance could be placed on such assistance, and that the Croton water must at all times be handled so as to give proper results independent of the Catskill water.

CHEMICAL APPLICATION

The chemicals used, and the average and maximum amounts were as follows:

For coagulation, aluminum sulphate, average 1 grain per gallon, maximum 2 grains per gallon. Lime or soda were to be used when necessary to neutralize excess carbonic acid in the filtered water; the quantities contemplated were: lime, maximum, 2 grains per gallon;

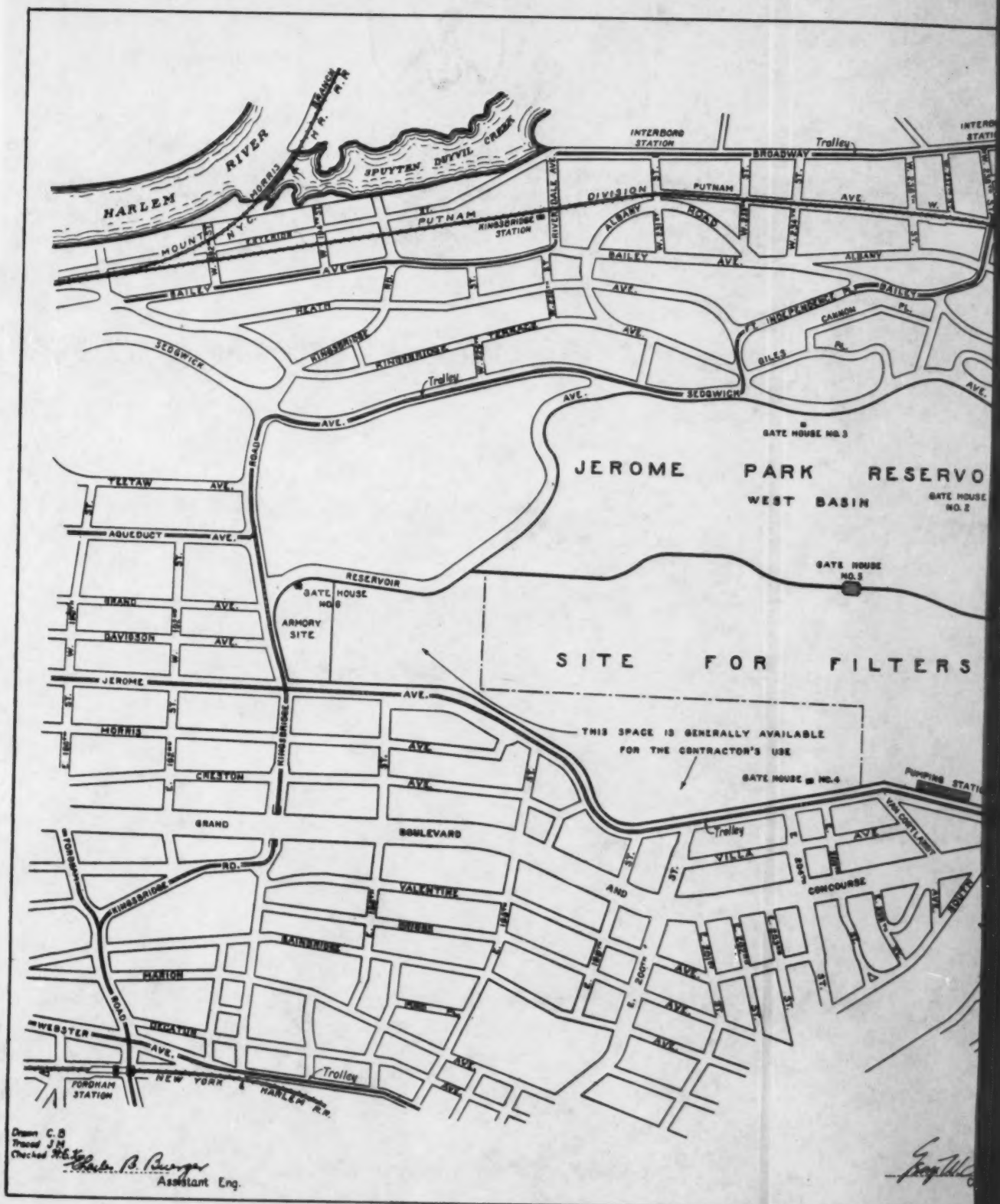
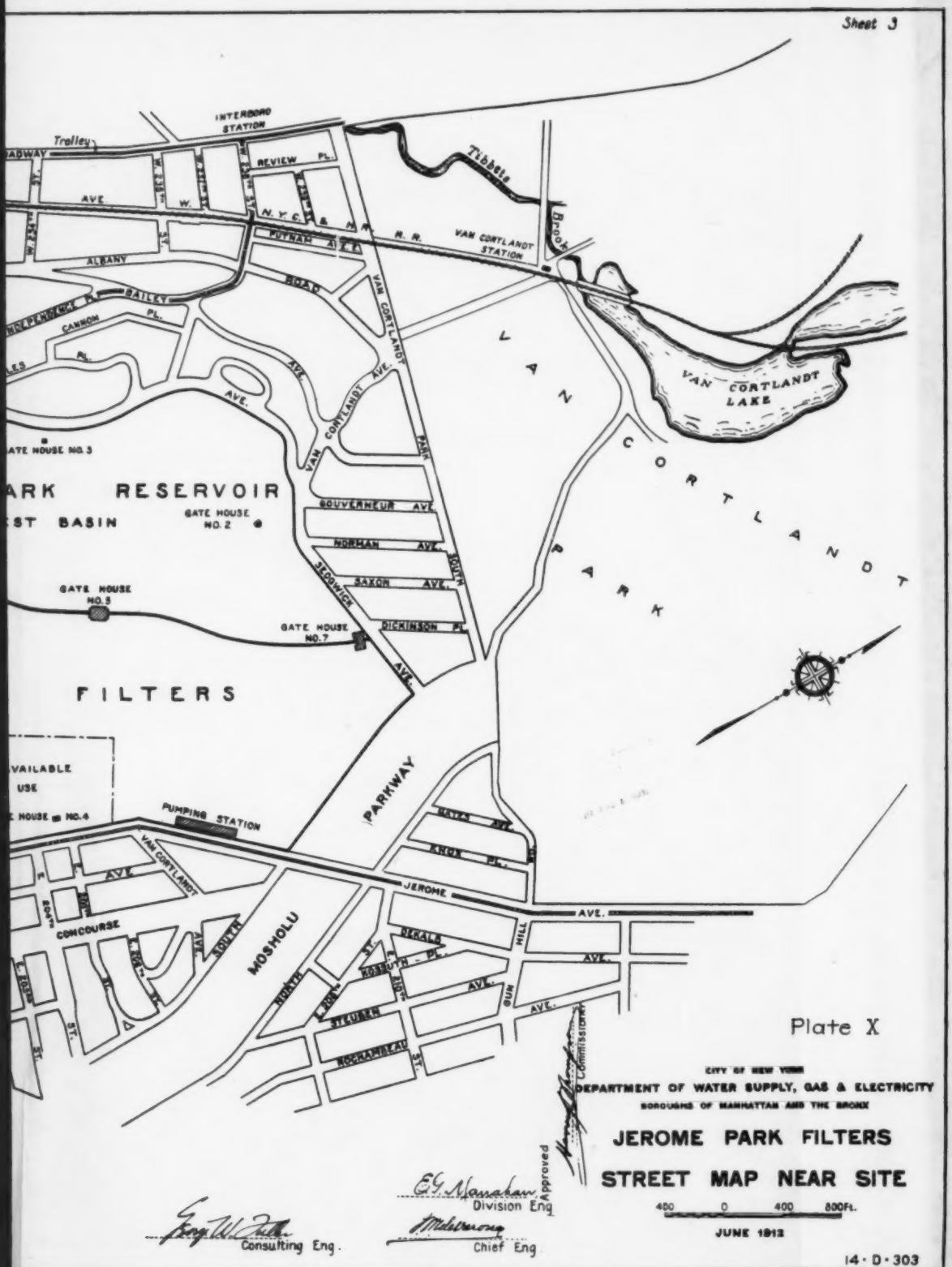
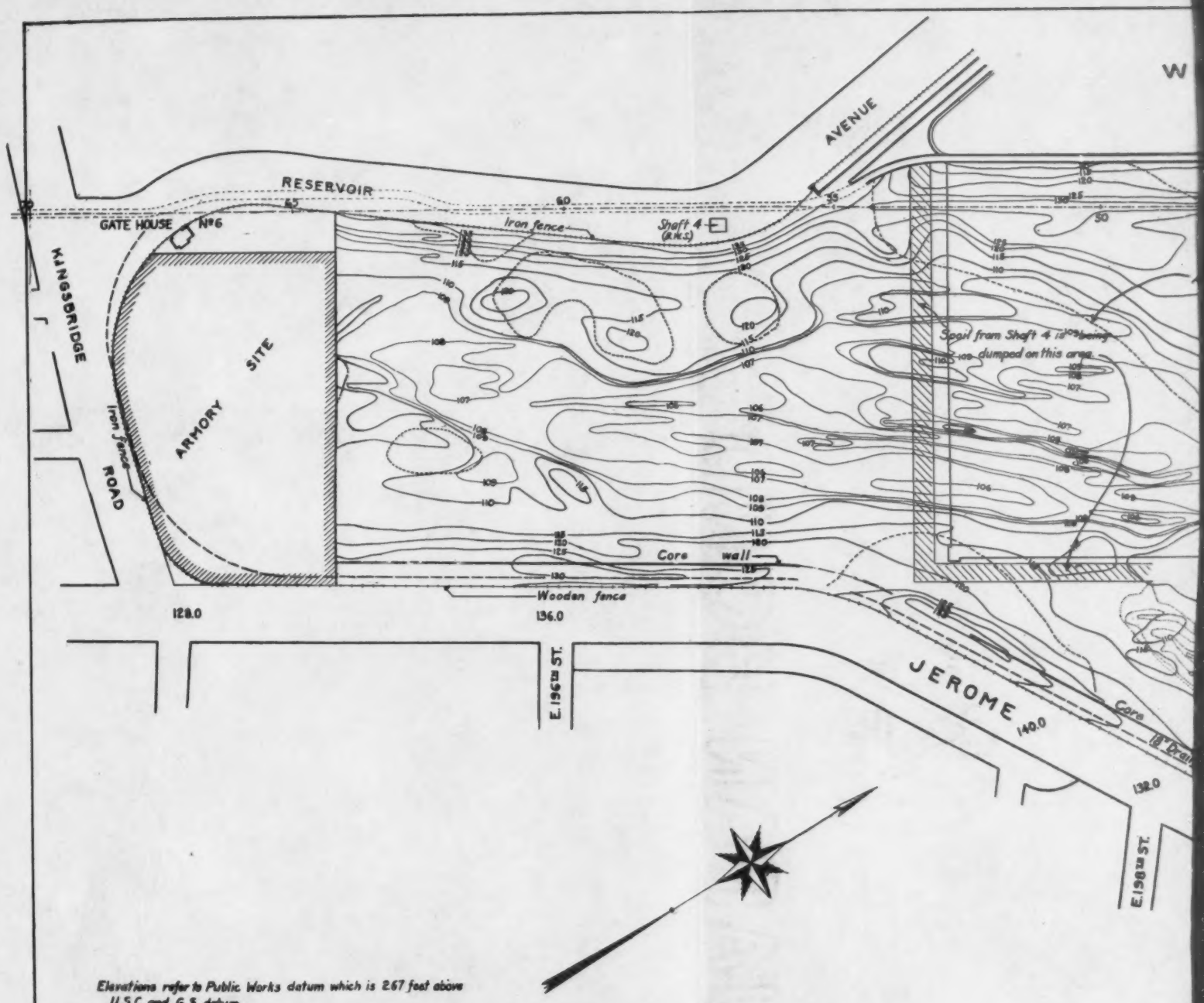


PLATE X. FILTER SITE



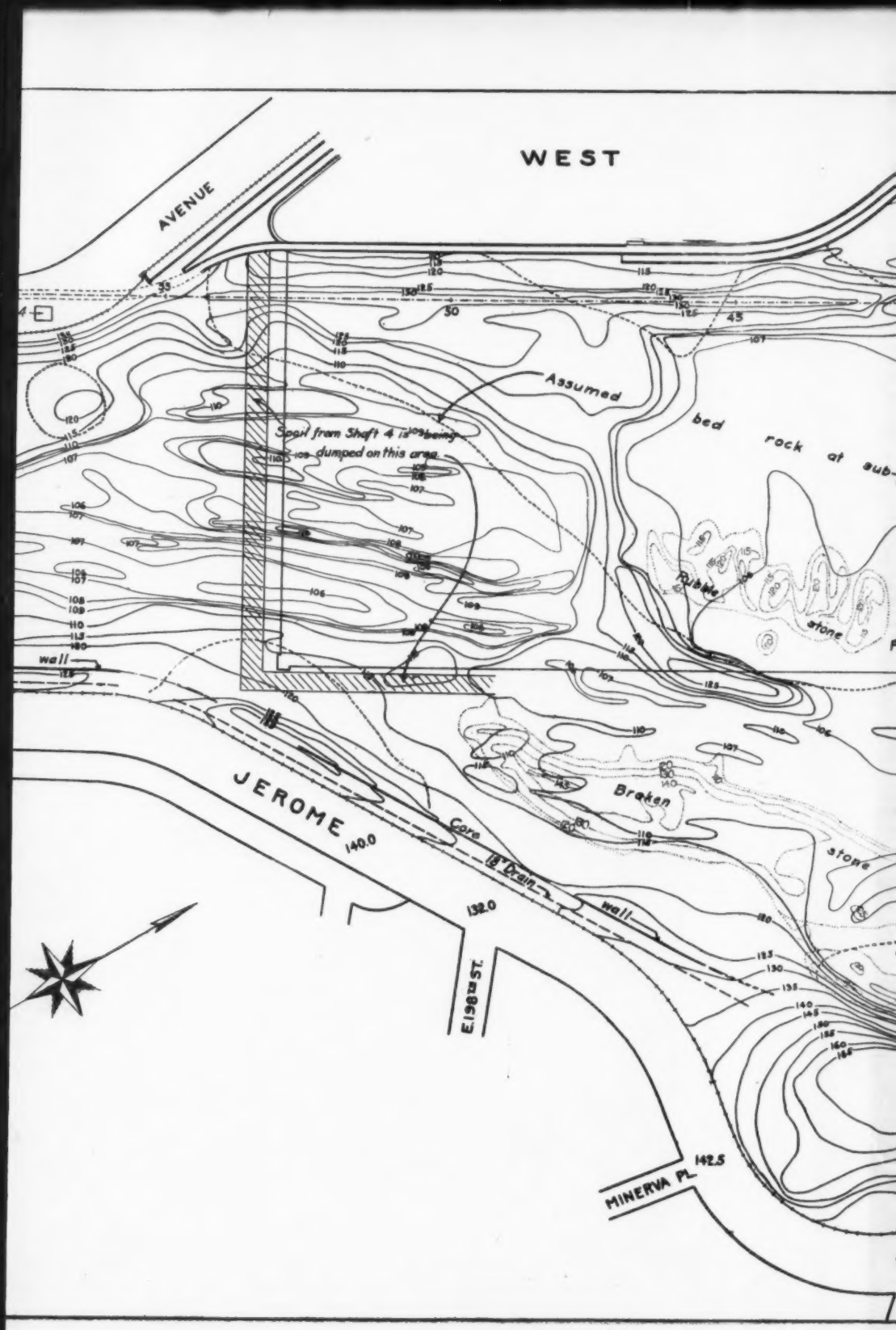
E X. FILTER SITE



Elevations refer to Public Works datum which is 2.67 feet above U.S.C. and G.S. datum.

Drawn A.R.N.A. & W.E.N.
Traced E.K.
Checked C.F.

Charles B. Burger
Assistant Eng.


WEST
AVENUE
Assumed
bed
rock at
Spoil from Shaft 4 is being
dumped on this area
Rubble
stone
wall
JEROME
140.0
Core
18" Drain
wall
Broken
stone
132.0
E.138TH ST.
MINERVA PL
142.5

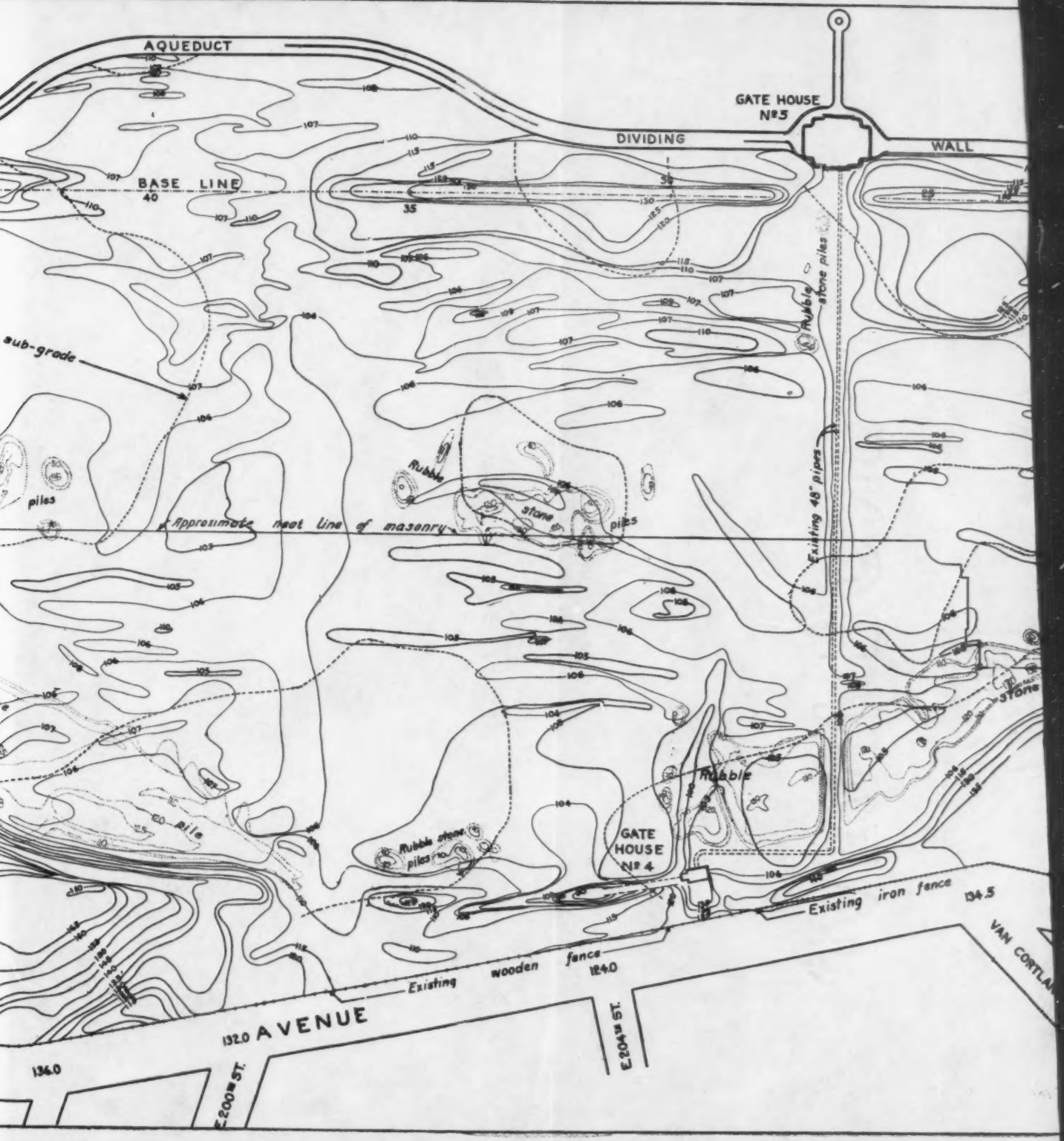


PLATE XI. TOPOGRAPHY OF FILTER SITE

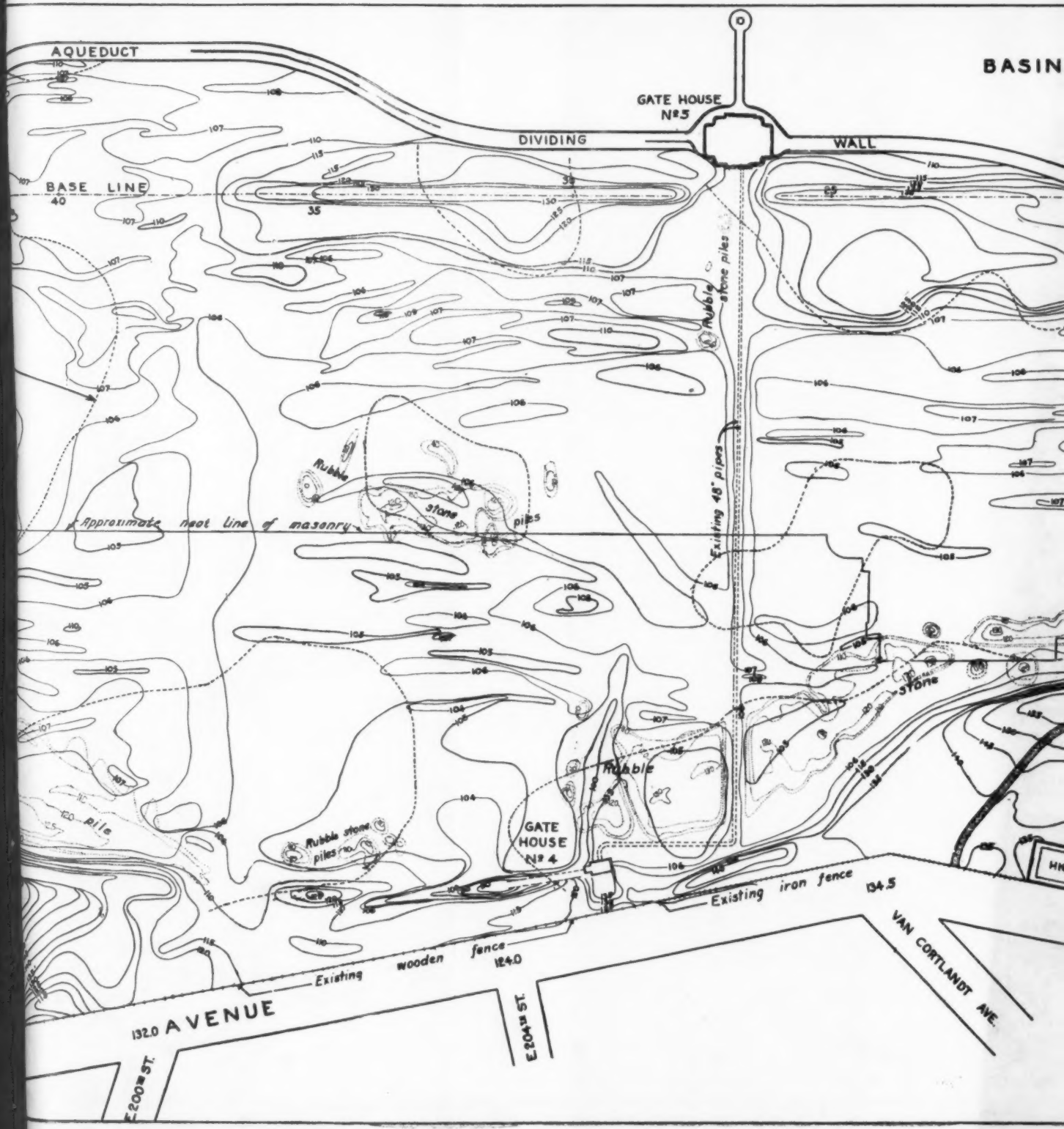


PLATE XI. TOPOGRAPHY OF FILTER SITE

IN

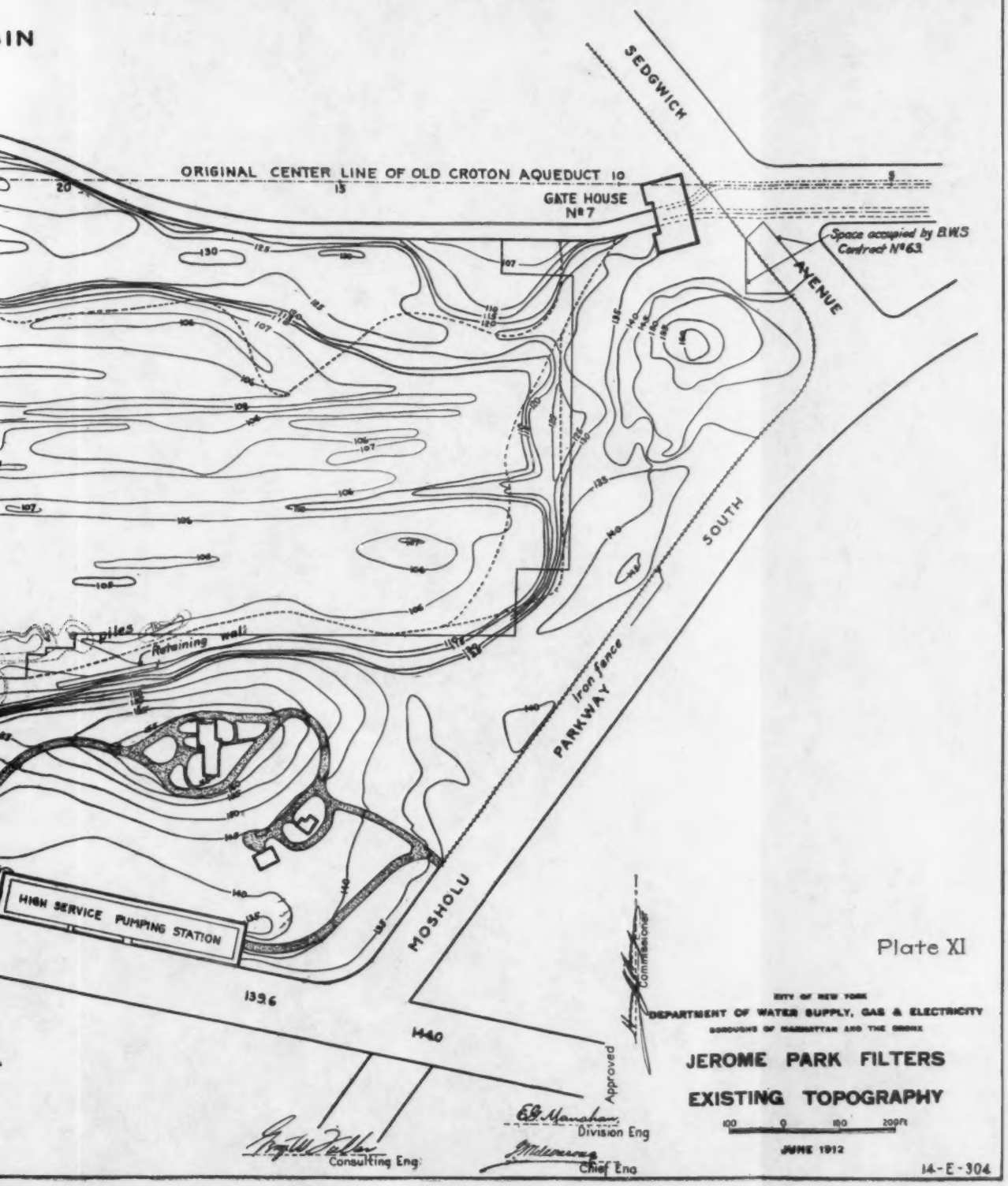


Plate XI

CITY OF NEW YORK
DEPARTMENT OF WATER SUPPLY, GAS & ELECTRICITY
BOROUGHS OF MANHATTAN AND THE BRONX

JEROME PARK FILTERS

EXISTING TOPOGRAPHY

100 0 100 200ft

JUNE 1912

Approved
Commissioner
E.G. Manahan
Division Eng
Chief Eng
Consulting Eng

14-E-304

TABLE 10

Croton aqueduct, hydraulic gradient

LOCATION	STATION	DISTANCE	SOFFIT OF ARCH.	HYDRAULIC GRADE LINE, AT RATES PER DAY IN MILLIONS OF GALLONS			
				293	300	310	320
		feet					
New Croton Gate House	4 + 75	0	153.53	156.92	157.87	159.36	160.86
Shaft 8, Pocantico River	453 + 84	44,909	147.58	150.37	151.05	152.05	153.05
Pocantico blowoff	495 + 30	49,055	147.03	149.76	150.42	151.36	152.32
Sawmill River Cut	732 + 72	72,797	143.85	146.30	146.80	147.48	148.20
Ardsley Blowoff	826 + 25	82,150	142.64	144.94	145.38	145.97	146.56
Shaft No. 14 A	830 + 57	82,582	142.58	144.88	145.33	145.91	146.48
Tibbet's Brook Blowoff	1121 + 25	111,650	138.73	140.62	140.91	141.16	141.46
Dunwoodie Cut and Cover	1140 + 30	113,550	138.48	140.33	140.60	140.85	141.12
Gate House 1	1266 + 18	126,143	136.78	138.52	138.67	138.76	138.86
Gate House 7		5,665	135.94	137.36	137.36	137.36	137.36
East Chamber Gate House 7				137.11	137.11	137.11	137.11
S in feet per 1,000 — Main Aqueduct				0.146	0.152	0.163	0.174
S in feet per 1,000 — Branch Aqueduct				0.222	0.232	0.247	0.265

Elevations given in Croton Datum which is 3.61 feet below Jerome Park Datum.

Main Aqueduct: $a = 155.624$ square feet, $r = 3.45$, $c = 130$.

Jerome Park Branch: $a = 132.63$ square feet, $r = 3.145$, $c = 130$.

soda (alternative) maximum 1.5 grains per gallon. Sterilization was to be effected by hypochlorite of lime applied at a rate of from 5 to 10 pounds per million gallons.

DECARBONIZATION

A good deal of study was given to the question of what is known as the "red water plague." The use of soft waters, particularly of waters from swampy sources, in iron service pipes usually brings into the water a large amount of red rust. This is particularly marked in hot water service pipes.

The presence of carbonic acid in quantity explains this action. An elaborate series of tests was made by the Department laboratories, details of which have been published in the 1912 Annual Report of the Department. The conclusions there arrived at are that the corrosion is directly proportional to the carbonic acid present; that alum-treated water produces practically no more rust than the Croton water untreated, mainly because the amount of carbonic acid set free is small; that it is desirable only to neutralize the free carbonic acid either with lime or soda.

It is well known, however, that complaints are more general on this score with filtered waters than with unfiltered waters. It is quite likely that this is to be explained by the action of the clean filtered water in removing existing protecting coats of deposits and keeping the iron surfaces somewhat cleaner.

To eliminate a possible source of complaint on the introduction of filtered water, it was decided to add lime or soda during the summer months when the carbonic acid content was high, and to reduce this carbonic acid to a point where no objectionable effect would be experienced.

DETAILS OF DESIGN

Plate XII shows the general arrangement of the plant as designed. The water supplied from both the old Croton aqueducts and the new Croton aqueduct is supplied at Gate House No. 7. The water can be sent directly to the east side to the filter plant, or directly to the storage basin on the west side, or else through the dividing wall south to the outlet at about station 46 + 00, delivering water at the far end of the storage basin, to create a desirable circulation.

The water from the Gate House 7 passes through a 14-foot Venturi meter with 7-foot 3-inch throat to Gate House 8.

Gate House 8 is a central control chamber, where the water may at will be sent through one-half or the whole of the mixing chamber and any or all of the settling basins. It is designed for that purpose with suitable dividing walls and thirty-four 5 by 12 sluice gates, operated by oil pressure cylinders. Plate XIII shows a plan of Gate House 8.

The mixing chamber has a capacity of 6, 200,000 gallons, or twenty-eight minutes normal flow, with nineteen turns at an average velocity of 1 foot per second.

There are five settling basins of an aggregate capacity of 82,000,000 gallons, giving a period of detention with all the basins in use of six hours, with a velocity of .40 of an inch per second.

Flushing and draining pipes and conduits are provided for both mixing chamber and settling basins.

A sixth basin is provided, with a capacity of 5,700,000 gallons for settling wash water, so that it may be refiltered at times of water shortage. This wash water is pumped at Gate House 8 back to the supply conduit.

The six basins all lead to Gate House 9 shown on the plan, similar to Gate House 8, but smaller.

The net area of the mixing chamber and settling basins was 11.6 acres, and the gross area occupied by these structures is 13.8 acres.

The coagulant is normally applied at Gate House 8 through a rubber pipe grid. Two 5-inch coagulant supply pipes lead to this point from the head house. For supplementary coagulation two 4-inch alum lines run to Gate House 9.

For emergency use one 6-inch alum pipe runs to Gate House 7 and means for dry application are provided at Gate House 7, consisting of a bronze cage or basket holding alum suspended in the current of the water.

All the basins are covered with a groined arch concrete roof and with 2 feet of earth cover.

The filters are completely separated from the settling basins as shown on Plate XII. There are 80 filters, divided into four series of 20 each, 10 arranged on each side of the operating gallery. The filters have an effective sand surface of 359 square feet per million gallons rating. A typical plan is shown on Plate XIV.

The feature of having the high water level in the reservoir 4 feet higher than the crown of the arches supporting the filters, and the necessity of adapting the design to all four possible conditions of

loading, viz., filter full and empty, and basin full and empty, required a special consideration of the basin walls, and these are materially heavier than is usual. Particular care is used to reduce friction because of the small head available; pipe velocities are uniformly low; it will be noticed that inlet and outlet openings are larger than the valves, with converging or diverging connecting pipes.

The filter sand specified is unusually course, 0.60 to 0.70 mm effective size.

The plans provided for three types of strainer system under alternate bidding items.

Type A consisted of cast-iron manifolds, cast-iron laterals with brass strainer cups for washing at the rate of 9 gallons per minute per square foot of sand surface, and a separate air system for air wash through cast-iron manifolds, and brass laterals, at the rate of 4 cubic feet per minute per square foot of sand surface.

Type B provided for the application of both air and water through the same system, using the Williamson strainer.

Type C consisted of concrete channels and perforated brass plates for applying water only at the rate of 15 gallons per minute per square foot of sand surface. The sand and gravel were separated by a brass wire screen.

Table 11 following gives the working velocities through the various parts of the three strainer systems for filtration and washing.

Wash water is to be stored in a reservoir on the hill top and intended to be supplied from the Catskill water system by gravity, and in case of emergency from the existing service pumping station.

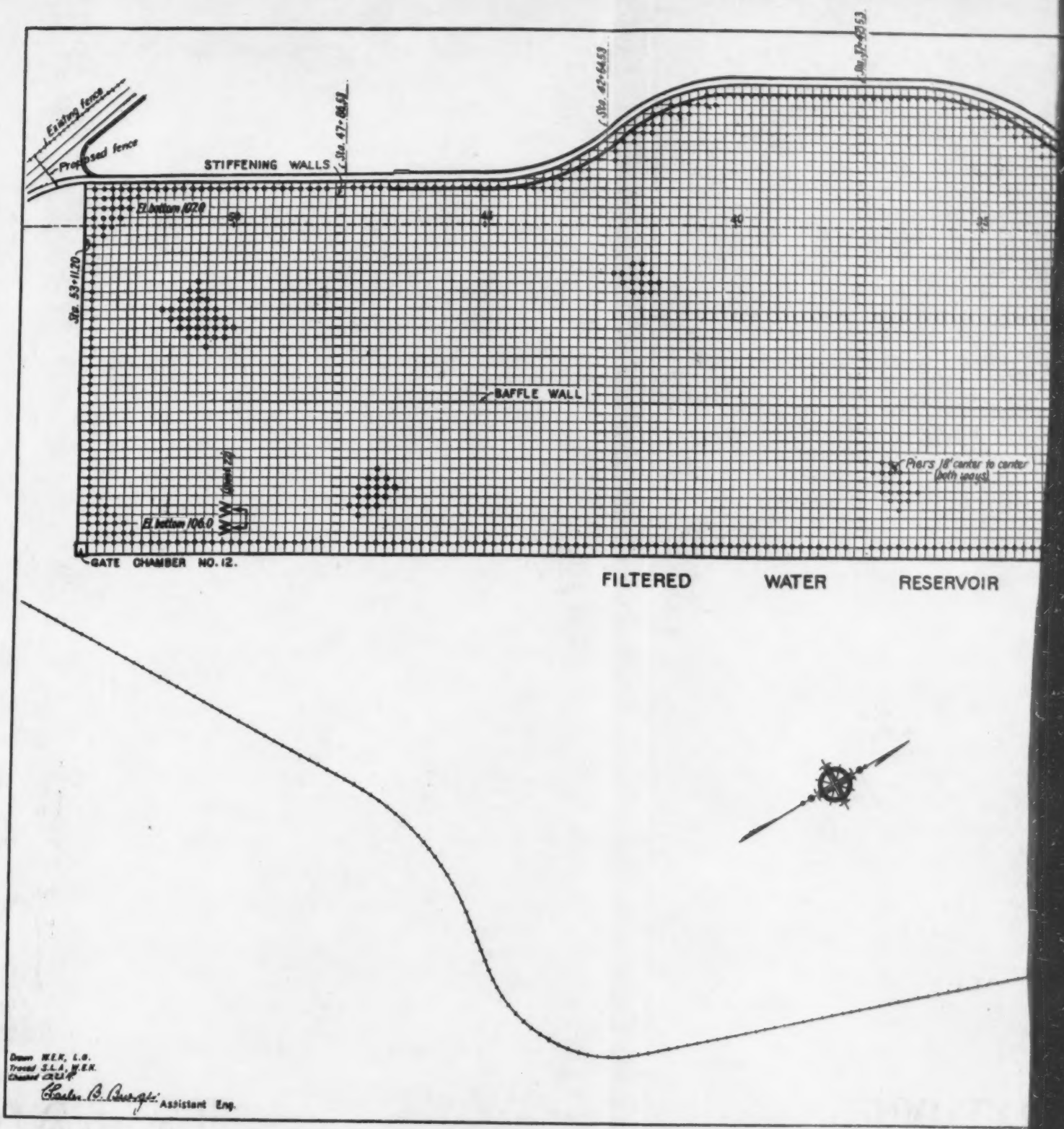
On leaving the filter galleries, the filtered water is treated with hypochlorite of lime.

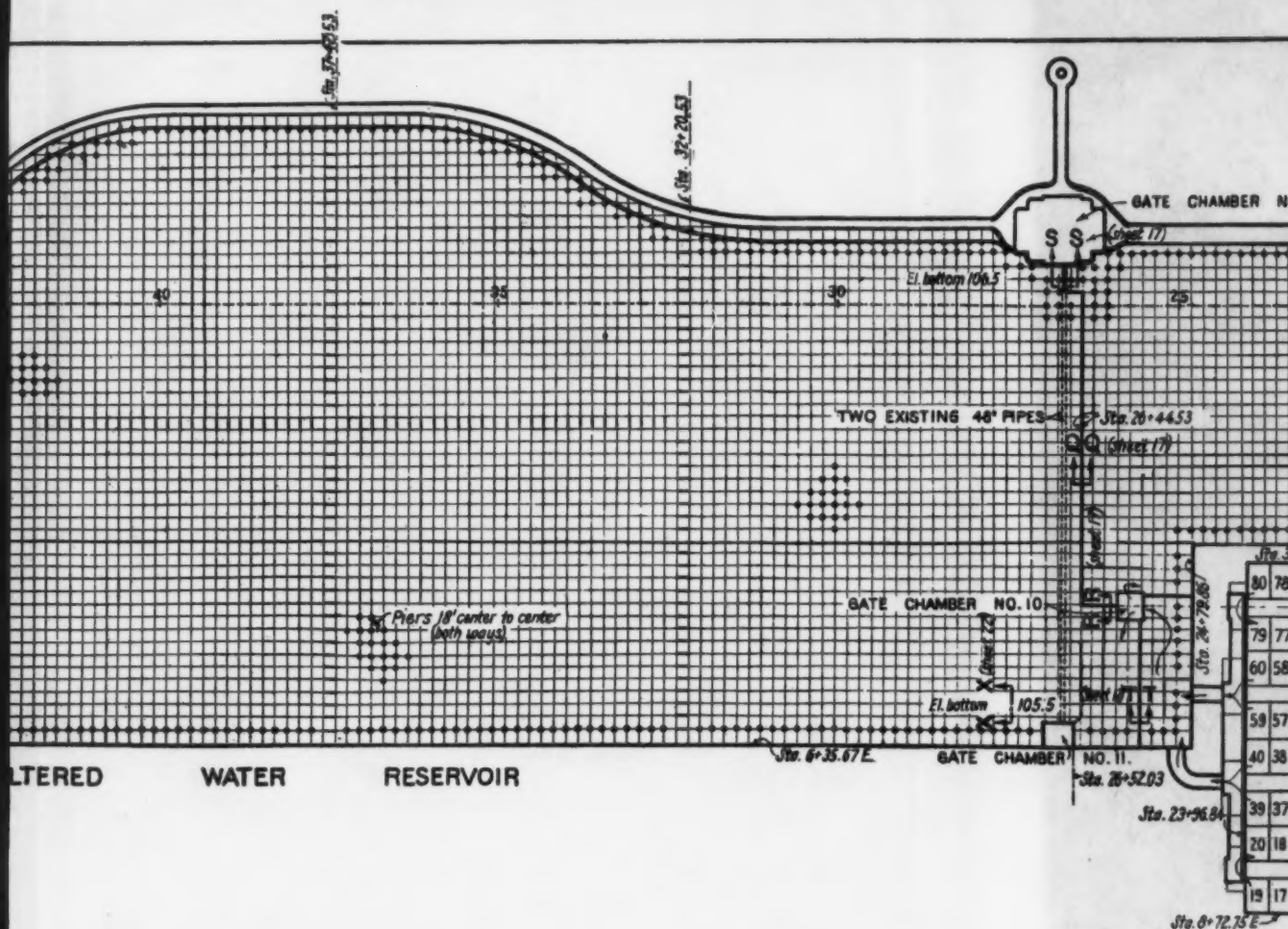
The net filtering area of the plant is 2.7 acres; the gross area occupied is 4.5 acres. The capacity of the reservoir under the filters is 14,400,000 gallons.

The south end of the four filter galleries is connected by a three-story building to be used as a laboratory for the department.

Plate XV shows a picture of the architectural treatment of the south facade; the remainder of the buildings were designed to harmonize with this front. Messrs. Hiss and Weeks were architects for this design.

The water, after receiving hypochlorite treatment, passed for a five-minute period through channels at a 1-foot per second velocity and then to Gate House 10 where, when needed, lime was to be applied.


Existing fence
Proposed fence
STIFFENING WALLS
Sta. 47+84.59
Sta. 47+64.59
Sta. 12+29.53
El. bottom 107.0
Sta. 53+11.20
BAFFLE WALL
W.W. 10 wide 7.9'
El. bottom 106.0
GATE CHAMBER NO. 12.
Piers 18' center to center (both ways)
FILTERED WATER RESERVOIR
Drawn W.E.K., L.G.
Traced S.L.A., W.E.K.
Checked C.E.S.
Charles B. Burdett Assistant Eng.



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reservoirs are subje
upon the character of

PLATE XII. GENERAL ARRANGEMENT OF FILTER PLANT

Sheet 2

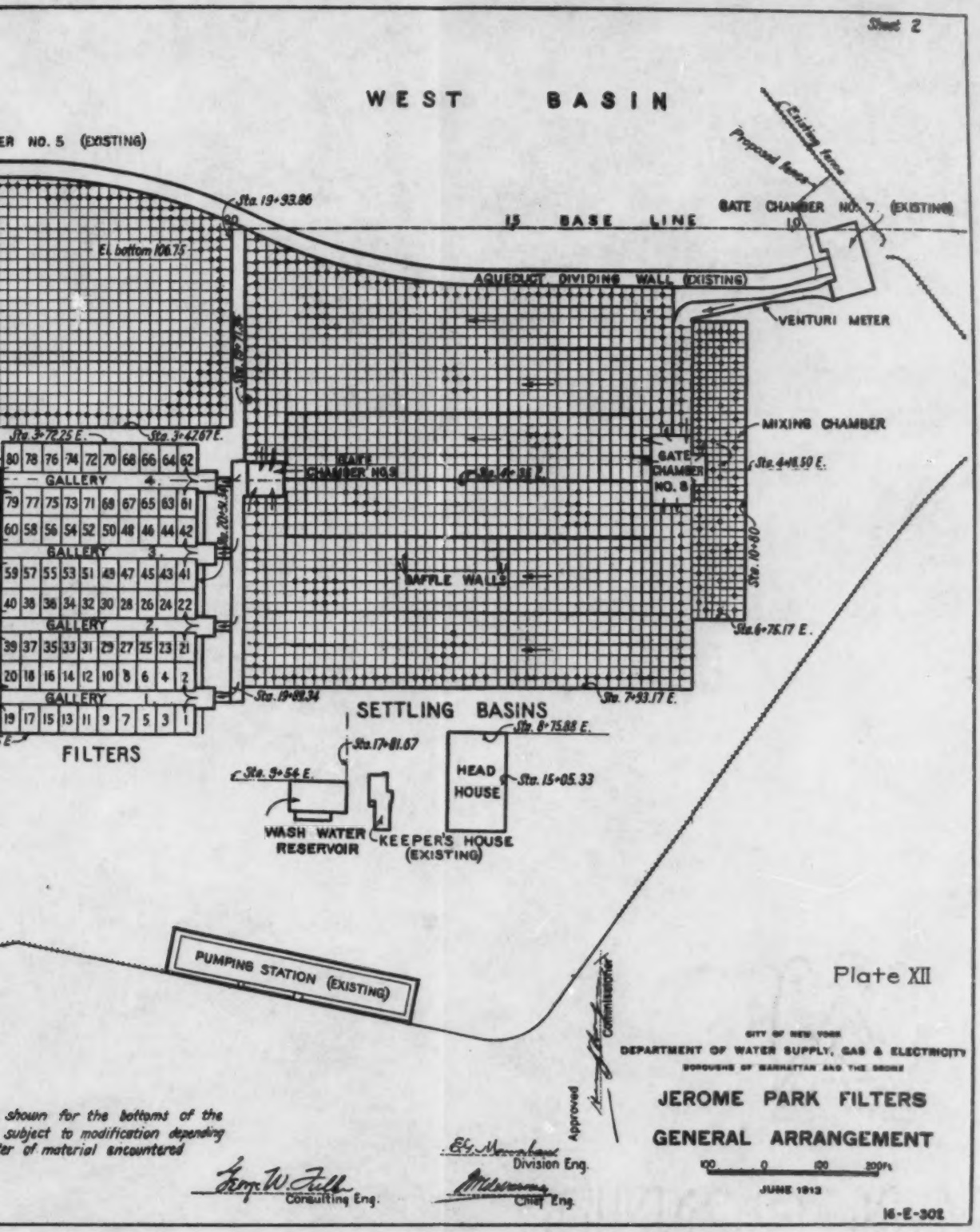


Plate XII

CITY OF NEW YORK
DEPARTMENT OF WATER SUPPLY, GAS & ELECTRICITY
BOROUGHS OF MANHATTAN AND THE BRONX

JEROME PARK FILTERS

GENERAL ARRANGEMENT

100 0 100 200 Ft.

JUNE 1913

16-E-302

shown for the bottoms of the
subject to modification depending
er of material encountered

Approved
Commissioner

E.G. Hawkhurst
Division Eng.

George W. Fuller
Consulting Eng.

Chief Eng.

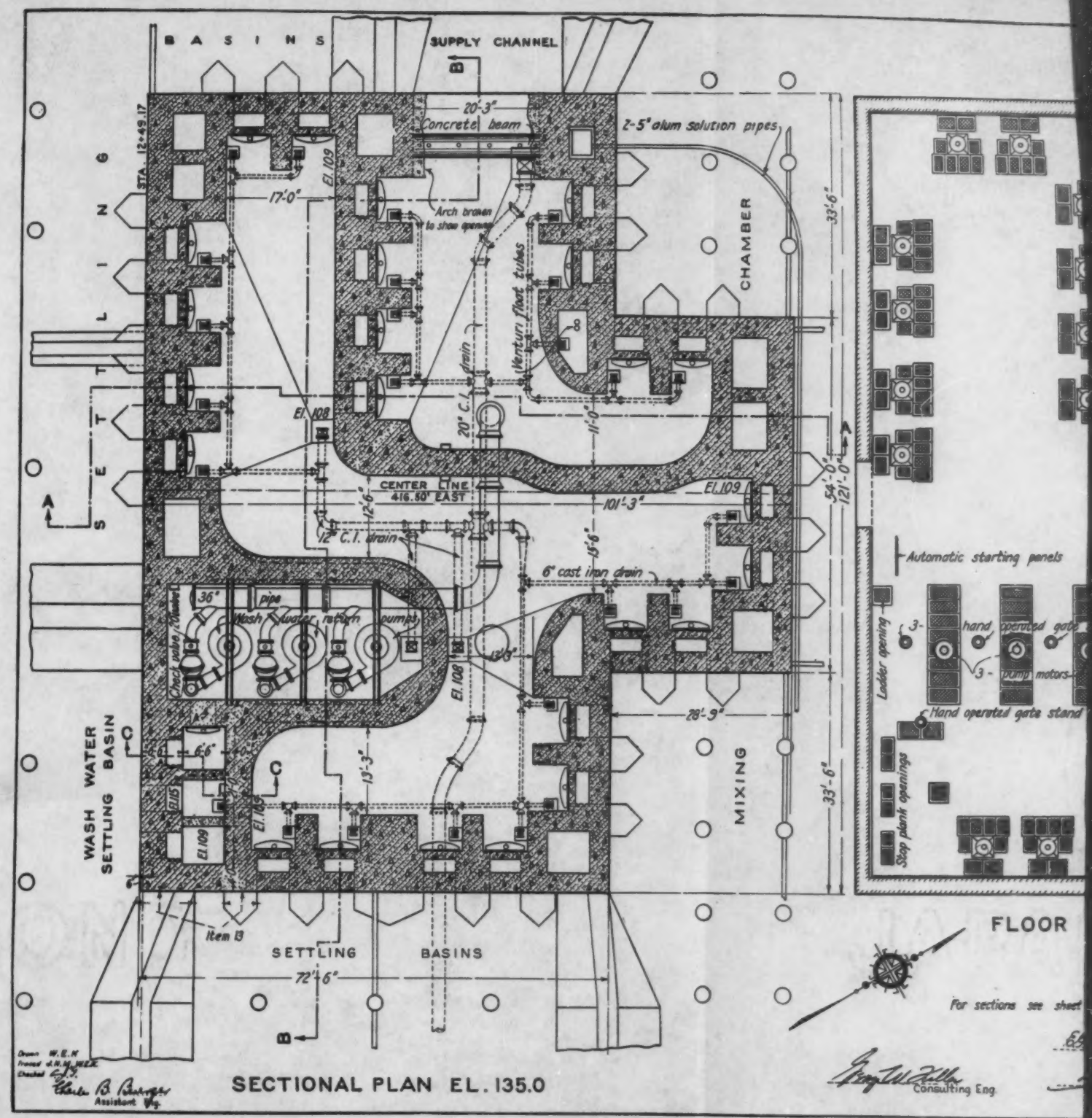
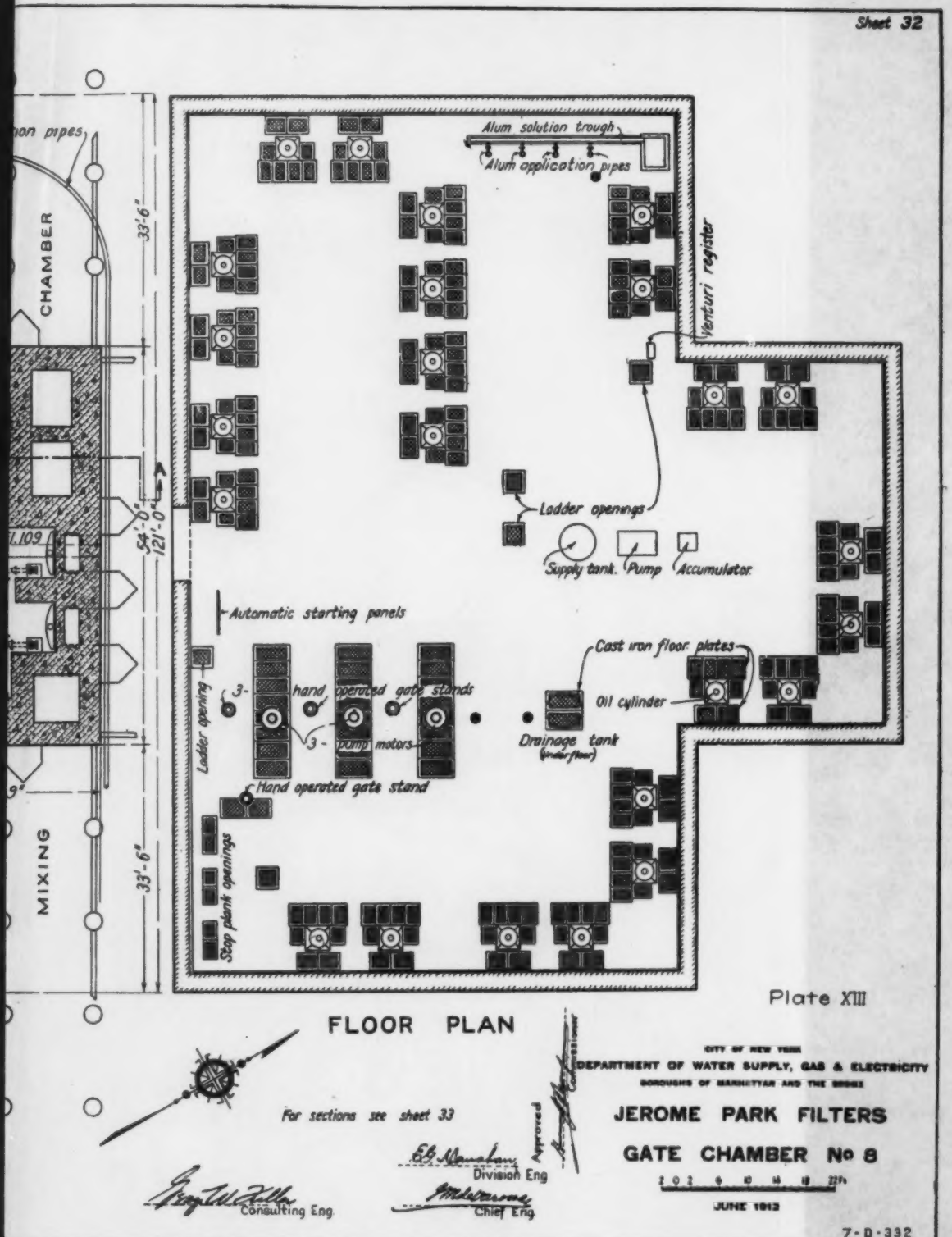
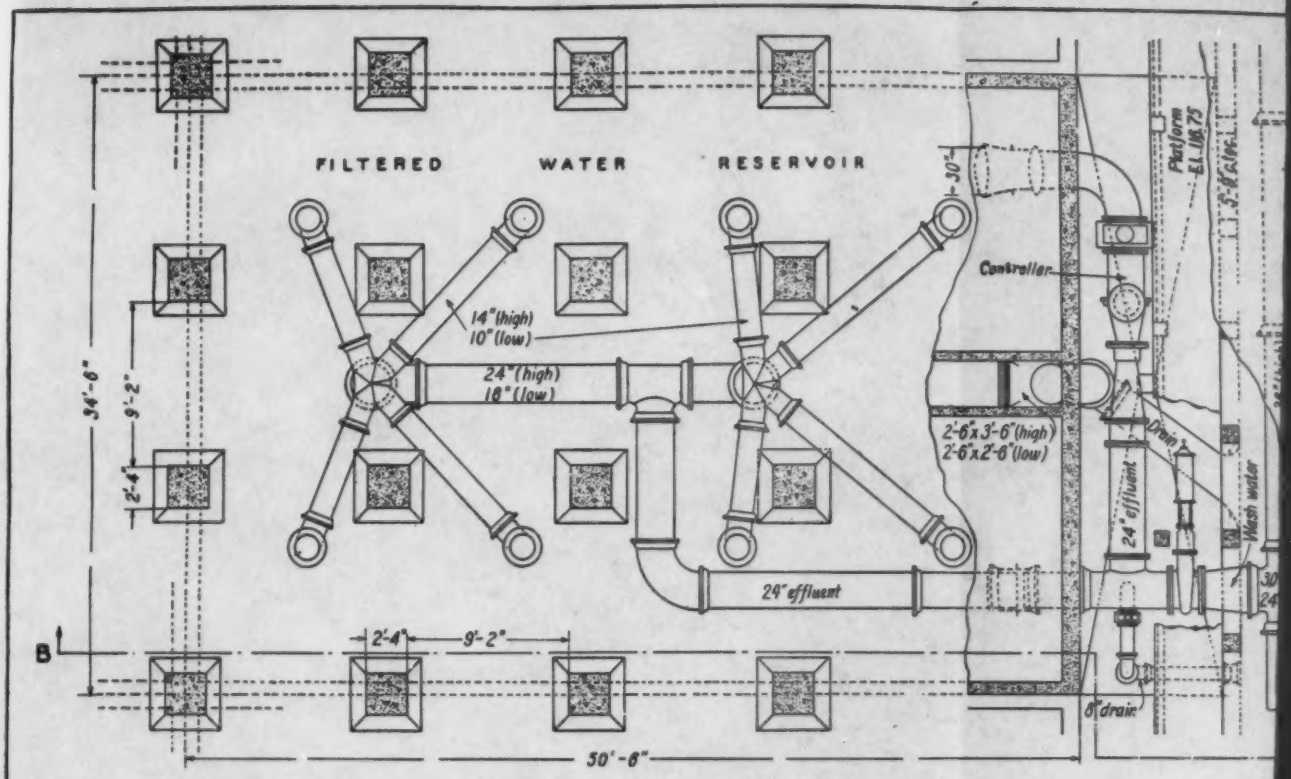


PLATE XIII. GATE CHAMBER NO. 8

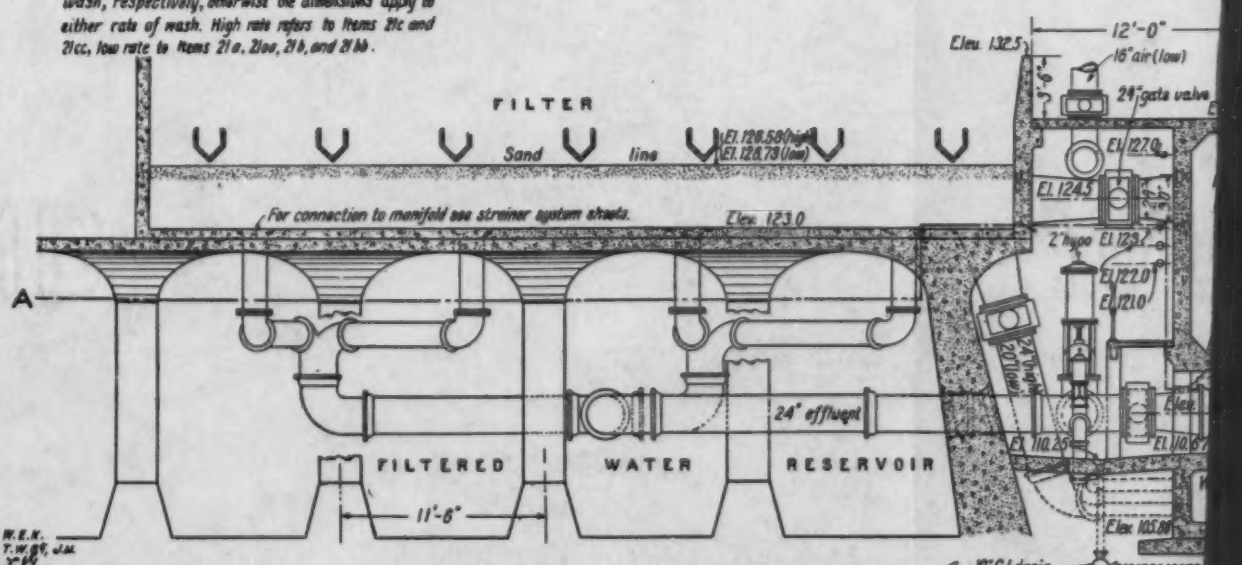


III. Gate Chamber No. 8



SECTION A-A

The pipe dimensions with the words (high) or (low) give pipe sizes required for high or low rate of wash, respectively, otherwise the dimensions apply to either rate of wash. High rate refers to Items 21c and 21cc, low rate to Items 21a, 21aa, 21b, and 21bb.

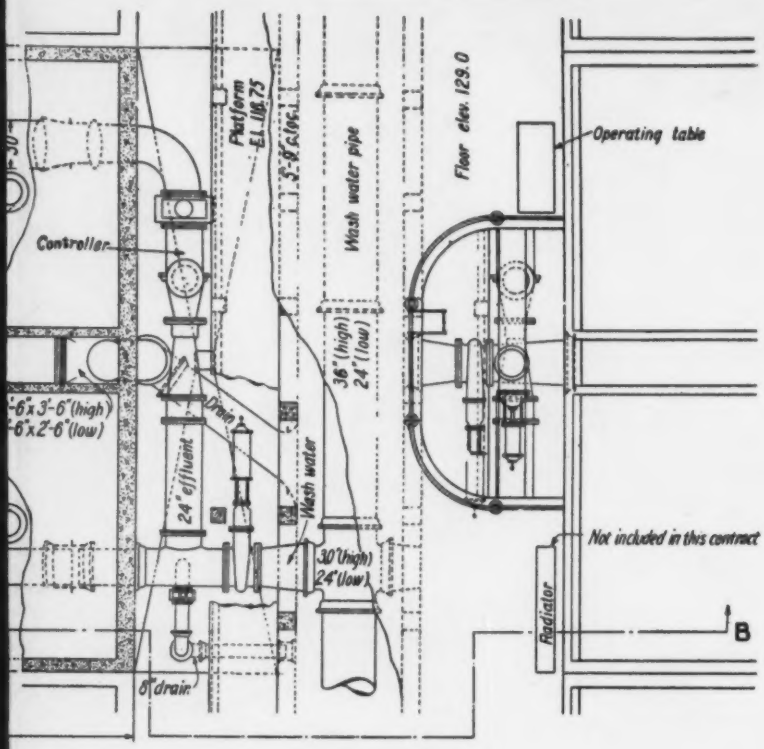


SECTION B-B

Drawn W.E.K.
Traced T.W.G.Y., J.M.
Checked C.H.D.
Charles B. Breed
Assistant Eng.

George W. Fuller
Consulting Eng.

PLATE XIV. SECTION OF FILTER AND FILTER G



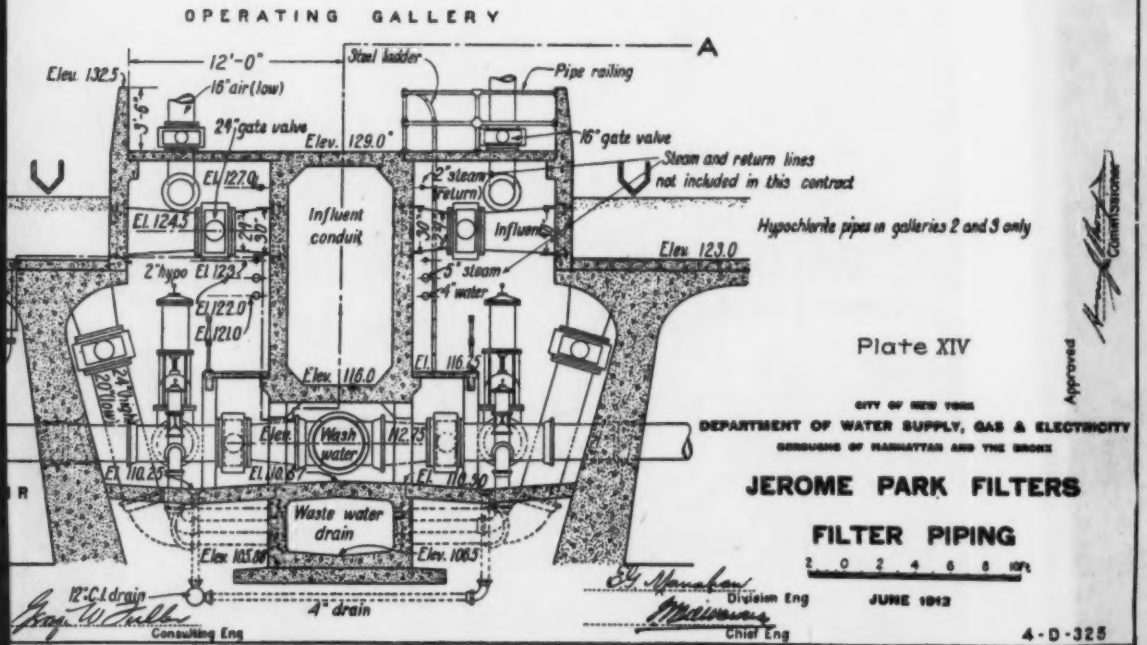


Plate XIV

CITY OF NEW YORK
DEPARTMENT OF WATER SUPPLY, GAS & ELECTRICITY
BOROUGHS OF MANHATTAN AND THE BRONX

JEROME PARK FILTERS

FILTER PIPING

2 0 2 4 6 8 10 Ft.

JUNE 1912

Approved
Commissioner

Division Eng
Chief Eng
Consulting Eng

4-D-325

N OF FILTER AND FILTER GALLERY

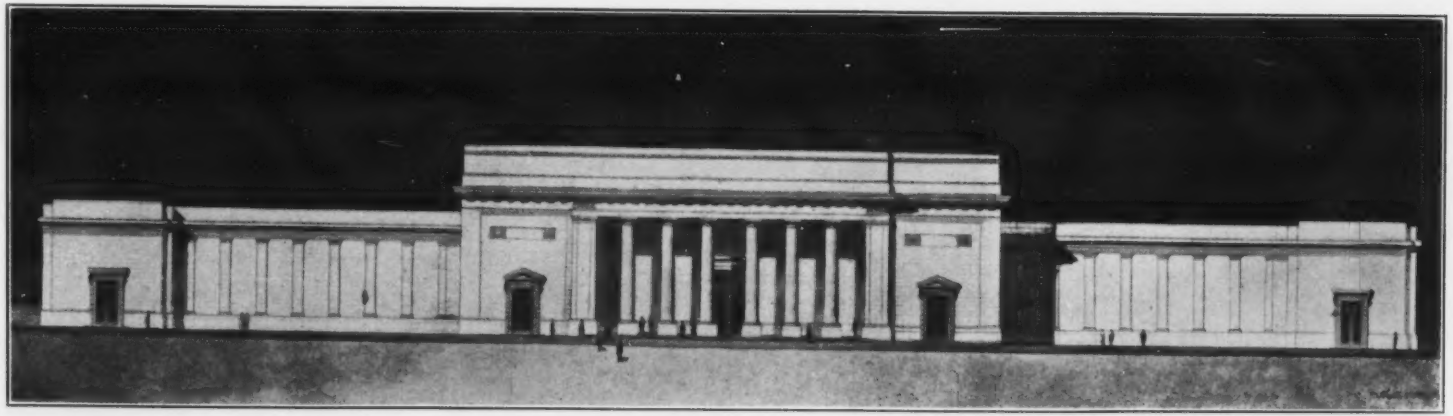


PLATE XV. SOUTH ELEVATION OF FILTER HOUSE

TABLE 11

Jerome Park filters. Strainer and air system, working velocities feet per second

	FILTRATION	WASH	AIR (FREE)
Type A			
Through sand	0.00443	0.02	0.0668
Strainer holes, or air holes	1.50	6.75	260.
Strainer neck	2.20	10.00	.....
Laterals	.71	3.18	86.
Manifold	1.11	5.00	79.
10-inch supply	1.46	6.60	.....
18-inch collector	1.80	8.15	.....
24-inch main	2.03	9.17	.....
16-inch air main	.....	.....	68.8
Type B			
Through sand	.00443	0.02	.0668
Strainer holes	1.55	7.10	23.4
Strainer neck	2.15	9.70	360.
Laterals	.58	2.60	8.68
Manifold	1.11	5.00	8.32
10-inch supply	1.46	6.60	.....
18-inch collector	1.80	8.15	.....
24-inch main	2.03	9.17	.....
16-inch air main	.....	.....	68.8
Type C			
Through sand	.00443	.0333	
Strainer holes	1.41	10.8	
Laterals	.26	1.98	
Manifold channel	.40	3.08	
14-inch supply	.73	5.60	
24-inch collector	.99	7.61	
24-inch main	2.00	15.2	

The filtered water reservoir was a covered two section basin of the usual type of a net area of 53.55 acres, a gross area of 55.13 acres and a capacity of 356,000,000 gallons.

The water from the filtered water basins passed through Gate House 5 to a shaft connecting with the pressure tunnel to the city.

The head house was located on the side of the existing hill, with elevations suitable to feed alum and hypochlorite by gravity from the feed boxes. A valuable novelty of design was the arrangement which permitted incoming trucks loaded with chemicals to enter the building at the highside so that they could discharge their contents

directly into the storage bins, from which the chemicals were discharged into the mixing boxes below, and no elevator was required. The capacities allowed for various chemicals are:

	cubic feet per million gallons per day
Alum storage and lime or soda storage	190
Alum solution tanks	108
Lime suspension tanks	42
Hypochlorite solution tanks	34

BIDS RECEIVED

The completed plans and specifications were submitted to the Board of Estimate on June 25, 1912, and were referred to their Chief Engineer, Mr. Nelson P. Lewis, and to the Principal Assistant Engineer of the Department of Finance, Mr. Chandler Withington. The report of these gentlemen is given below.

On the 23d instant you handed us the plans, contract and specifications for the Jerome Park filters, with a request that we examine them carefully and report to you as to their adequacy and sufficiency to secure the desired result, namely, the proper filtration of Croton water by a system of mechanical filters.

We have carefully examined the plans, the contract and the specifications, and have conferred with the officers of the Department of Water Supply, Gas and Electricity and the Engineer who was in immediate charge of the preparation of the plans, all of whom have readily given us any information we desired.

We have suggested a few minor changes in the form of contract in order to remove any ambiguity and to eliminate what might prove to be slight inconsistencies. These changes the Commissioner of Water Supply has approved and has incorporated in the form of contract. We have also suggested certain modifications in the specifications with a view to making them more definite, and have advised certain changes in some of the incidental work, such as the building of roads, which we believe would give better results without any increase in cost. These changes met with the approval of the Engineers and have also been incorporated in the form of contract and specifications which we herewith submit.

In general, the specifications are in our judgment admirable in form and arrangement and exceptionally free from ambiguity, and we can see no reason why they should not be approved by the Committee.

The plans and specifications for the first contract were printed and advertised and after some delay bids were received on January 23, 1913. Nine bids were received ranging from $5,139,015 to $6,752,000. The engineers' estimate was $5,916,700.

A comparison of prices on some of the more important items is given in Table 12.

TABLE 12

Contract prices

ITEM NO.	DESCRIPTION	UNIT	ENGINEERS' ESTIMATE	AVERAGE BID	LOW BID
1	Earth excavation.....	cu. yd.	$0.70	$0.87	$0.74
2	Foundation excavation...	cu. yd.	1.00	1.42	.84
3	Roch excavation.....	cu. yd.	1.75	1.53	.98
8	Puddle.....	cu. yd.	4.00	3.37	3.20
11	Foundation concrete....	cu. yd.	7.75	6.80	6.50
12	Pier and vaulting concrete.	cu. yd.	9.50	8.81	7.90
13	Wall concrete..	cu. yd.	7.75	7.43	6.50
14	Thin reinforced concrete.....	cu. yd.	13.50	12.84	10.60
20	Main drain...	lump	50,000.00	70,718.44	65,000.00
21	Filter equipment.....		1,024,000.00	1,343 695.00	1,043,000.00
22	Cast-iron pipe..	ton	45.00	46.00	39.00
23	Specials.....	ton	90.00	87.78	85.00
25	Sluice gates...	lump	100,000.00	91,056.00	82,000.00
26	Gate valves...	pound	.10	.102	.09
	Total....		$5,916,700.00	$6,058,354.00	$5,139,015.00

NOTE: Concrete prices are affected by the presence on the ground of considerable suitable broken stone and the availability of excavated stone for this purpose.

OPPOSITION TO THE PROJECT

At this time a strong opposition to the project manifested itself. The controversial features have been admirably covered in the articles in the *Engineering News* of the dates May 22, May 29, and June 5, 1913, and need be referred to at this place only briefly.

A small number of detailed criticisms were made of the plans and specifications. As these have been examined and unreservedly approved by more than a dozen competent and disinterested engineers including the first Board of Consulting Experts, the Board of Advisory Engineers to the Stock Budget Committee of the Board of Estimate and Apportionment, the Chief Engineer of the Board of Estimate, and

the Principal Assistant Engineer of the Department of Finance, these criticisms may be summarily dismissed. Criticism was also made of the site selected, and the method of filtration recommended.

The whole problem was referred by the Board of Estimate to an advisory commission of engineers, composed of Messrs. Nicholas S. Hill, Jr., Consulting Engineer; John H. Gregory, Consulting Engineer; E. P. Goodrich, Consulting Engineer to the President of Manhattan; Amos L. Schaeffer, Consulting Engineer to the President of the Bronx; John H. Frazee, Assistant Engineer, Department of Finance.

The commission investigated all phases of the matter for three months, and in April, 1913, submitted two reports, the majority report by the first four members listed above, and the minority report by Mr. Frazee, the Comptroller's assistant.

The majority report may be briefly abstracted by the following quotations:

Quality of Croton water. As at present delivered to the consumer, the Croton water would be characterized by the water analyst as noticeably colored and slightly turbid, at times quite turbid; at other times containing numbers of microscopic organisms with an odor persistently vegetable and occasionally aromatic, grassy or even fishy; reasonably soft; a good boiler water and generally satisfactory for industrial purposes; ordinarily safe but at times sufficiently polluted to indicate the possible danger of infection from water borne diseases.

Evidence of the unattractiveness of the Croton water as supplied to the consumer is found in the large number of local filter plants and in the common use of bottled water.

This condition (of pollution) suggests a potential danger which is a serious menace to any community.

The value of long storage is thus evident. Rainfall and runoff records show that in times of flood the storage in Croton Lake is only sufficient to provide for a retention of the water flowing therein for from 10 to 15 days. As the time required for the water to flow through the aqueduct from Croton Lake to New York may be measured in hours, the period of detention in times of flood is not sufficient to give security against the introduction of an infected water into the City of New York, provided infection exists on the watershed.

The typhoid fever statistics of New York indicate that the Croton water is a reasonably safe supply from the sanitary viewpoint, except possibly for short intervals, as indicated by the typhoid outbreak in 1907.

Germicidal treatments. Hypochlorite of lime has been found to be less efficient in the treatment of raw water than filtered water, except raw waters in which there is little or no organic matter. When applied to raw water with an appreciable organic content, larger doses must be used because enough must be applied to combine with the organic and other matters in the water which are capable of rapid oxidation in its presence, and still leave enough excess to perform its germicidal function. The amount of organic matter in many waters

fluctuates rapidly from day to day, and even from hour to hour, and the difficulty of regulating the dose of hypochlorite to meet these fluctuations probably accounts for the incomplete sterilization of raw water.

Jerome Park filters. The location of the proposed filter plant in the east basin of the Jerome Park reservoir has been found to be the *best* location for the installation of the plant.

The mechanical filtration plant, as specified in the plans of the Water Department, in contradistinction to the slow sand or modified sand filtration plant, is in the opinion of the commission the best to install.

The detail design and general construction of the filter plant as proposed are in accord with modern practice, and are carefully and well executed. The specifications are reasonable and sufficient, and have developed competition, as shown by the number of bids received for the construction of the filters.

The Water Department estimates that the total cost of construction will be $7,727,000. Independent estimates prepared by this commission check within 6 per cent of the total cost of $7,727,000.

It (the annual cost of filtration) will amount to about ¾ of a cent per $100 in the tax rate.

Offsetting these costs will be the saving effected by numbers of property owners who are now required to filter, or otherwise treat, the water supplied to them by the City of New York. It is a recognized fact that the great majority of hotels and some apartment houses, as well as many private residences and office buildings, resort to local filtration of the Croton water. In addition to these costs, some taxpayers bear an indirect burden due to the purchase of bottled water for drinking purposes, a practice which is very common and may continue to some extent even after filtered water is introduced.

It is the opinion of this commission that the filtration of the Croton water supply is not imperative, but that it is highly advisable to filter the water at this time both with a view to improving the physical quality and as a safeguard against potential danger, and we so recommend with the exception of one member dissenting.

The dissenting report of Mr. Frazee was to the following effect:

I am of the opinion that it would be as satisfactorily efficient at this time, and far more economical, to strike at the sources of trouble, some of which were recognized as existing ten years ago, but toward the abatement of which nothing has been done.

The advancement of sanitary and medical science in recent years in the development of this treatment (hypochlorite application) and the inocculation against typhoid, tend strongly to discount the conjectural possibility of future outbreaks pending delay and particularly so if the polluting sources known to exist in part and to be indicated are abated.

The factors of bacterial content, which may affect the health, and of color and turbidity, taste and odor, harmless and offensive in the aesthetic sense only, all exceed at times the standards set by experts for the filtered standard. All these undesirable features are intensified beyond the necessities by the condition existing within the city and on the Croton watershed.

Water is cumulatively affected by passing through the Central Park reservoirs, through mains more or less obstructed by sediment, vegetation and organisms, to be finally stored in house tanks on the higher structures, which are pumped full at night and drawn from in the daytime.

The power of the State Board of Health should be invoked, or legislative action be sought, to compel the railroads to use special unleakable sanitary equipment during the time of crossing this territory (the Croton watershed). In any case protective pans should be installed over all waters.

It is to be noted, further, that at least certain of the highway bridges crossing tributary water have the ordinary type of plank flooring, through the crevices of which B. coli from the excreta of farm animals can readily pass or be washed by rains.

This wash water (from farming lands, highways, etc.) might be isolated by dikes and be led to points where sedimentation could be effected or preliminary filtration had.

Reforestation, shrub and grass planting to give a sand binding, absorbent root system and to provide a screen from highway dust, will ameliorate these conditions.

The Commission on Additional Water Supply noted the improvement to be effected by the removal of sediment (from swamps and shallow flowage), and the draining of the swamps. Nothing has been done in relief, whereas dredging or filling in would effect beneficial results.

But the responsibilities warrant strengthening in numbers that a system of patrol can be instituted by tests in accordance with an uptodate sanitary map's requirements. The sensitiveness of scent of dogs would warrant their training as auxiliaries.

Mr. Frazee further figures on the financial advantages of delay. He assumes that the land proposed as a site would be sold; that 10 lots to the acre would be taxed, bringing a tax revenue of $330,000 per year; that $3,000,000 would be obtained from the land, that a saving of $400,000 in operation would be occasioned by substituting his palliative measures for filtration. Capitalizing his various assumed savings, he figures that in the 50 year bond life, a delay of only five years in the initiation of the filtration plant will save the city $31,000,000.

Commenting first on this financial statement, it is worth noting:

1. That if, as the majority report states, the site selected is the best one, the land cannot properly be sold.

2. That an assumption that the cost of remedial measures in the watershed will save $400,000 in operation rests on an imaginary foundation. It is a fact that watershed protective methods as suggested to improve the quality of the water will cost more both for investment and maintenance than filtration.

The difficulties and cost of effectual watershed protection have not been fully realized. An investigation was made by the Department of Water Supply at the request of Comptroller Prendergast and expense estimated, as follows:

Prevention of pollution from railroad trains:		
Panning bridges	$ 16,000	
Embankments	546,000	562,000
Highway bridges		103,000
Aeration		225,000
Control of microscopic growths:		
Draining of swamps, 10 square miles at $75 per acre	480,000	
By-passing reservoirs	750,000	
Reduction of shallow flowage, 5,500,000 cubic yards at $0.25 per cubic yard	1,375,000	2,605,000
Reforestation of watershed, 11,000,000 trees		360,000
Treatment of road wash, Dikes for 925,000 feet highways	3,530,000	
Treatment of cultivated lands	1,194,000	
Land cost for marginal protecting area, 26,000 acres at $600	15,600,000	20,324,000
Total		$24,179,000

The annual fixed charge alone at 5.13 per cent is $1,240,000.

In connection with the estimated costs of protecting land, it is interesting to note that the city now owns 21,000 acres, costing $12,000,000, or $600 per acre. Of this, 12,000 acres is reservoir area, and 9,000 acres protecting and marginal area. An addition of 26,000 acres for adequate protection would not be a high estimate of the land required.

The annual taxes are now about $100,000, practically $5 per acre per year. The additional taxes on this protection land alone would amount at this rate to $130,000 per year in addition to fixed charges of $1,240,000, and with no allowance for additional maintenance.

CLEANING OF RESERVOIRS AND MAINS

In 1907 the south basin was emptied and found to be practically clean after 12 years service. Mr. D. D. Jackson reported:

> An examination of the old reservoir, south side of Central Park, was made on September 26, 1907, and in most portions of the reservoir no mud was present. In some sections there were small deposits, the largest being just south of the intake, apparently formed by an eddy at that place. Results show that it would be unnecessary to draw down the reservoir for the removal of this comparatively small amount of mud.

It is to be noted, too, that on an average the water is improved in sanitary quality by passing through the reservoirs, and through the mains. It is not improved in number of microscopic organisms, as these will grow in storage, and no reservoirs or main cleaning will materially help the water in this particular. The by-passing and covering of the reservoirs would be of assistance.

The two reports of the Advisory Commission of Engineers were submitted to the Board of Estimate early in May, 1913. On May 8, 1913, the Citizens' Union made a further protest. Passing minor criticisms of no importance, it was charged that the original report of the Board of Experts had been suppressed, and the intent of the Board misrepresented.

This accusation was submitted to the Chairman of the Board, Mr. Allen Hazen, who wrote in explanation to the Commissioner, from which I quote as follows from the *Engineers News* of May 22, 1913:

> It is in my opinion necessary to filter Croton water. It is necessary to filter it because in no other way can a supply of clean water, of good quality, worthy of the city, be secured.
>
> Filtration will do more to improve the quality of the water and protect those who use it from infections resulting from pollutions of the catchment areas than has been accomplished by all the measures of sanitary inspection and prevention of pollution of every kind that have been applied up to the present time, and that can be reasonably applied in future.
>
> It is also necessary to improve the physical characteristics of the water in order to make it acceptable for drinking water by the people of the city.

A hearing on this last phase of the matter, the desirability of filtration from the standpoint of health protection, was held by Mr. Mitchell, now Mayor. The opponents of the plan were represented by Major Cassius E. Gillette, Mr. R. Walter Cruezbar, Mr. Alexander Potter, Mr. Charles L. Parmelee, Mr. John H. Frazee.

In support of the project appeared Mr. Allen Hazen, Mr. J. Waldo Smith, Mr. Nicholas S. Hill, Jr., Col. Merritt H. Smith, Mr. George A. Johnson, Mr. George W. Fuller.

Reference was made to testimony of Mr. Fuller and of Mr. Johnson in 1909, that the Jersey City Water supply would be safe if treated with hypochlorite.

This was answered by both of these gentlemen to the effect that the 1909 testimony was based on information then available; that present information showed that hypochlorite treatment, when applied to

water containing large quantities of organic matter, was of variable and uncertain efficiency; that particularly where the amount of such organic matter varied, hypochlorite application could give no safe dependence, and was not an adequate substitute for filtration.

The last chapter of this controversy was written on May 22, 1913, when Mr. Mitchell reported to the Board of Estimate his deduction that in his judgment hypochlorite would make our water safe, and that aeration would serve to remove from 50 to 75 per cent not only of the tastes and odors, but also of the color and turbidity from the water. With this he cast the vote which killed the project.

CONCLUSION

It is rather curious that a ten year long attempt to give the citizens of New York a drinking water supply of a quality such as is elsewhere demanded should have finally proven abortive. And even more curious that the same officials who had voted to give New York a proper drinking water should later with the same information before them reverse themselves, that a Board of Aldermen, who but a short time before were considerng a vote of censure on the executive department for their slowness in providing filtered water should a few months later repeal their ordinance and condemn the project; that the newspapers which had just stopped printing attacks on the city officers for their negligence in furnishing bad water, should write to oppose the so-called steal when they were about to receive their demands.

The various phases of this matter have been clearly before the city officials for many years. It has never been represented as absolutely imperative. Neither is it imperative for the city to clean its streets daily, to maintain good smooth pavements, or to build a $15,000,000 office building. But it is certainly desirable that a city of the rank of ours should have a pride in doing these things; and it should have a pride in drinking a water for which it need not apologize as reasonably safe, at most times, and fairly decent looking at most times.

Real safety we are not getting. Limited by the need of adding only enough hypochlorite not to spoil the taste, and with an organic content in the water varying from day to day, it is not physically possible closely to vary the hypochlorite dose so as to oxidize both the organic matter and the bacterial content and leave no objectionable excess. And conditions may arise at any time when the hypochlorite treatment will not be effectual in preventing the transmission of disease.

From an aesthetic viewpoint, our water is much of the time unobjectionable, but with turbidity running at times as high as 100 parts per million, color 60 parts per million, and sufficient microscopic organism to give unpleasant tastes and odors, our water supply is far from being a source of civic satisfaction.

This is not before the public as an engineering question. A comparison of the engineers mentioned above approving of the project with the list of those who have opposed, is convincing.

As a question of expediency, when a vote for $100,000,000 for good roads, which means an annual charge alone to the people of this city for interest and repairs of $5,000,000, in addition to the cost to the upstate population, is carried without hesitation, it is hard to understand a reluctance to spend $8,000,000 for drinking water. For maintenance and operation the cost to those served is but about 25 cents per capita per year, much less than is now the cost of bottled water to the public.

When this question comes up for final settlement, and no settlement is final until it is right, it is to be hoped that the legislative authorities will consider the question in all its aspects; and after they have satisfied themselves, and have appropriated the necessary funds, that they will leave the execution of the project in the hands of their engineers, to give the city a water supply which, instead of being at best not immediately dangerous, will be up to the standard maintained by ordinarily progressive communities.

The writer takes pleasure in acknowledging his indebtedness for data furnished, to Mr. I. M. deVarona, Chief Engineer, Mr. E. G. Manahan, Division Engineer, and Mr. Frank E. Hale, Director of Laboratories, all of the Department of Water Supply, and for assistance in writing this paper, to Mr. Charles B. Buerger, formerly in charge of design of the Jerome Park Filters.

DISCUSSION

MR. D. W. FRENCH: Having heard but a small part of Mr. Fuller's paper, am sure there is nothing the speaker can say that will be of interest or that would have any bearing in this particular project.

The filtering of public water supplies is a subject that we are all interested in and is getting to be quite the proper caper.

The Hackensack Water Company are now filtering from 28,000,000 to 30,000,000 gallons per day, the plant has been in operation for

nine years, and our Company do not feel that they ever made a move which met with so much satisfaction and popular approval as when it decided to filter the entire water supply.

We have felt very thankful since, that the movement was a voluntary one on our part, and not one into which we had been forced by public clamor, due to a typhoid fever outbreak.

The cost of filtering on the Hackensack system compares favorably with what is being done at other plants where a like quantity, and quality of water is being treated, but regardless of the plant or the amount of water being filtered daily, or the cost of filtering per million gallons, if the quality of filtered water is such that the company or municipality supplying it feel that there is no reasonable possibility of typhoid resulting from its use, then to my mind, filtering of any supply is worth all that it costs, or in other words, it is cheap at any price.

DR. WM. P. MASON: Mr. President: Side-stepping the matter to which you refer, let me say that in Troy we have no filtration, not because we do not believe in it, but because we have not the wherewithall to pay for the same. There is no question whatever but that Mr. Diven and the rest of us would like to see a filter plant in Troy and we all believe that we are going to have one, but Mr. Diven does not mention the date. Our cash is a little short at the present because we have other things to attend to, among them the present pipe line. It runs only part of the way to the city, and it is old and full of holes and it leaks, in fact it sprinkles a good part of the county, but we are in hopes that it will stay together sufficiently long to let us get a new one. We are very firm believers in filtration up our way. We have had designs made for a plant; they are on paper and old enough to be somewhat improved today if we should see any money in sight. Just at present we are believers rather than practicers only because we haven't the cash to put into a plant which we sadly need.

MR. J. M. DIVEN: Dr. Mason says Troy cannot at present afford a filter plant; but, can it afford not to have one? Of course, there must be the water to filter, and it is difficult some times to get that through a county wide "automatic sprinkler line." But the new pipe line is authorized, and the filter plant should be. Dr. Mason, doubtless, drinks bottled water, as do most of the well to do citizens of Troy, but the less fortunate have to drink such water as the city

furnishes, and this should by all means be the very best possible to supply to them. The bottled water consumed in Troy costs its citizens a considerable sum of money each year; have heard it estimated at $75,000 per annum, more than enough to pay interest on cost and depreciation on a filter plant and the cost of operation. Pure water should not be a matter of dollars and cents, it is a necessity and should be supplied at almost any cost.

It is a matter greatly to be regretted that two such important and influential bodies as the water commissioners of the greatest city in our land and the conservation commission of the leading state of the union should take such decidedly retrograde action as, the one stating that filters were not needed, as the water was good enough as it is; the other that permission should not be given to the city of New York to acquire additional water rights and lands for their preservation and protection to insure a water supply for the future. There is no "good enough" in water, even the very best must fail of perfection in some essential, but no effort should be missed to have the best possible under existing local conditions. The contention that only immediate or even near future supplies should be provided for is ridiculous and so great a body as the New York State Conservation Commission should be ashamed to have taken such a stand. It is the plain duty of all municipalities or water companies to secure, and secure *now* all the available water rights and water sheds that might at any time be utilized in improving and extending its water supply. Get these rights and lands first; for when taken up for power or other purposes it will be difficult and expensive to acquire them. One would think that the city of New York had already had sufficient experience of this kind. The lands and water rights acquired need not be now taken from industrial uses, but can be leased for any purpose that they might be useful for, but should first of all be available for the water supply of the city when needed, and so disposed of that they can be converted to city uses without long and tedious, not to mention expensive, legal proceedings, when needed; proceedings that might long delay a much needed additional water supply.

The example of two such influential bodies, examples that are apt to be copied or used as a precedent by others, is greatly to be regretted on account of the wide influence that it may have.

Mr. Nicholas S. Hill, Jr.: The speaker heartily concurs in all that Mr. Fuller has said, he is rather glad of the opportunity to say for

the Second Commission that it was unfortunate for it that they were not able to bring in a unanimous report on the question of the advisability of filtering the Croton water. One member of that Commission, however, had a conviction, and he could not see the immediate necessity of filtration, although he admitted that such a necessity might arise in the future. The majority of the Commission was satisfied that the present treatment with hypochlorite of lime was not sufficient to safeguard the supply. The wording of our report was that immediate filtration of the Croton water was highly advisable, although not imperative. We felt that this statement truly expressed the situation in the City of New York.

The typhoid fever death rate does not indicate the presence of endemic typhoid conditions. In fact, the typhoid death rate is less than the average. We were fully aware of the potential danger of an epidemic which may arise at any time if conditions on the watershed are ripe. In view of the tremendous population using the Croton water, this supply is a greater potential source of infection than any other single medium in existence, and we felt that if an epidemic did occur it might be of more alarming proportions than any hitherto known.

The Commission, moreover, felt that the physical quality of the Croton water did not measure up to the standards of the water supplies of other large cities of the United States and that filtration was justified on account of the high color and turbidity which are present at times and due to the offensive odors which exist at certain seasons of the year as a result of microscopic growths in the reservoirs. We felt, at least the majority did, that while the situation was not an imperative one in which danger was imminent, it was highly advisable to filter the water and to commence filtration operations at as early a date as possible.

In view of the fact that our Commission was requested to investigate alleged defects in the plans and specifications for the proposed Jerome Park filters, it gives me pleasure to say that we were unanimous in reporting that the site chosen for the filtration plant was the best location; that mechanical filtration as recommended by the consulting engineers was best suited to the needs of the Croton water; that the detail design and general construction of the filter plant as proposed were in accord with modern practice and were well and carefully executed, and that the specifications were reasonable and sufficient.

DR. FRANK E. HALE: The question of filtration has come up and been turned down. Filtration probably will come when the people demand it.

In lieu of filtration, we might run rapidly over the various means which are taken to protect the Croton supply. In the first place there is continual inspection by the Sanitary Division under Messrs. Coffin and Culyer. There are regular trips over the various watersheds, streams, reservoirs, etc., to observe physical signs of pollution, and these are abated.

Every case of typhoid fever on the watershed is recorded and carefully looked after. Disinfectants are furnished and if necessary the cases are removed to the hopsitals. All trains in passing over the watershed have their toilets locked during transit. Bridges over the reservoirs and streams are swept daily to remove the horse droppings, etc.

At several points on the watershed the sewage is treated and besides at several small places chlorination is used. There is a disposal plant at Brewsters with subsoil irrigation. At Mount Kisco there is an entire sewage disposal plant, comprised of a septic tank, contact beds, sedimentation and filtration beds, followed by chlorination, and the raw sewage goes through the plant and comes out as clear as crystal and at times safe to drink.

In the Laboratory Division samples are taken weekly from the various sources of the supply and analyzed and also daily from the supply as it leaves Croton Lake, as it arrives at the city at 135th street, and further down in the city at City Hall. Between Croton Lake and the city, at Dunwoodie, the whole Croton supply is continuously chlorinated.

Probably the greatest trouble we have with the Croton water is the taste and odor coming from microscopic organisms. We try to avoid this by drawing the water at a depth wherever possible and by treatment locally with copper sulphate. The speaker might say that the reservoirs are so deep that amounts are figured on the top 10 feet only in treating the water. We have also tried the last couple of years to improve the supply by drawing from Croton Lake at depths, in 1912 about 50 feet below the surface, and during the last year about 75 feet below the surface.

During 1912 the thermocline of which Mr. Fuller spoke was some 15 to 20 feet below the surface. During last summer there were two thermoclines, one about 20 to 25 feet and the second at 90 feet.

Below 90 feet the water has been completely stagnant, but above 90 feet the draft at a depth has kept the water in circulation.

Here are a few figures to show what the effect of drawing at a depth has upon the supply: In 1911 the temperature during the months of July, August and September at 135th Street, that is, of the water as it reached the city, was 72°; in 1912 it was 67°; and in 1913, 65°. The improvement during these three months was 7°. At times the water was at least 10° cooler than in previous years.

The number of microscopic organisms in the water at 135th Street in 1911 was 2836; in 1912, 1072; and in 1913, 677; in other words, less than 25 per cent of what it was two years previously. That this is due to drawing the water from the bottom is shown from the figures taken at Croton Lake. The surface of Croton Lake in 1913 had 1405 organisms against the 677 in the water at 135th Street. The samples taken in the screen chamber of the gate house where the water is just starting for the city contained 635 as compared with 677 organisms at 135th Street, showing that this benefit did come from drawing the water at a depth.

The treatment of the supply with chlorine at Dunwoodie seems to have been quite effective during the last year. As the water left the gate house at Croton Lake there was an average of 7 per cent in 0.1 cc., 23 per cent in 1 cc. and 59 per cent in 10 cc. tests for B. coli. When it reached the city it contained 0 per cent in .1 cc.; 3 per cent in 1 cc.; and 23 per cent in 10 cc. Comparing the tests by multiplying the 0.1 cc. tests by 100, the 1 cc. tests by 10, and adding to the 10 cc. tests, there was 95 per cent removal of B. coli. In the same way during 1912 there was 90 per cent removal of B. coli. This reduction was probably caused chiefly by the chlorine treatment.

Mr. Hazen: It is apparent that the filtration of the Croton water is primarily a question of giving clean decent water. The sanitary side of the question is not the dominating one at the present time.

James M. Caird (by letter): This paper by Mr. Fuller is most interesting as it deals with the large amount of detail sanitary work which is being carried on to protect the quality of the New York City water supply.

The fact is also noted, that notwithstanding all of these excellent methods of treatment and patrol the quality of the water is still such that it should be considered suspicious.

Conditions in this country are becoming such that the time is not far distant when all surface waters will have to be filtered, as is the case in some foreign countries.

It has been repeatedly shown that even with the most elaborate system of inspection of the watershed it is at times impossible to exclude dangerous pollution, which is eventually followed by epidemics.

It is surely wise to maintain a water as free from pollution as possible so that when it is filtered results cannot help being highly satisfactory.

The public judge the quality of a water by its appearance and so long as the water is free from color and turbidity, they are satisfied.

The use of some spring waters which are satisfactory in appearance is some times very dangerous.

A large percentage of the population are unable to purchase spring water, and these certainly should be supplied with pure water.

Hotels and some other buildings have filtration plants, but why should not the entire population be served with pure water?

It is certainly strange that the installation of a filter plant should be delayed when it is considered that its installation and operation would cost about 25 cents per year per capita, equal to about 2 cents per month. When it is considered how much improvement in general health would follow the use of a strictly pure water this would more than offset the small cost.

With a municipal filter plant in proper operation, the use of individual filters would cease and all manufacturers, which require pure water, would also save money.

While the typhoid fever death rate is not high when compared with American cities, yet it is much higher than in some foreign cities where the water supply is properly filtered.

It is not the intention to convey the idea that all of the typhoid fever is due to the water, but an improved water supply would certainly cause an improvement in the general health and therefore less chance of contracting typhoid from sources other than water.

If this supply was furnished by a corporation how long would the public stand for a water containing a turbidity as high as 100 parts per million; color 10 to 65 parts per million; bacteria 5 to 1200 per cubic centimeter (agar counts twenty-four hours at 37.5° C.) not to mention the odor and taste due to algae?

Filtration will remove some of the odor and taste due to algae as will also aeration but unless the algae are killed by treatment they

will cause serious trouble in operating any filter plant as well as great expense.

Mention is made that the water in the reservoir, at times, is treated with copper sulphate to kill the algae. The dose being calculated for a depth of 10 feet, but no mention of the strength of this dose is made.

Dr. Hale shows that B. Coli-communis is present in 3 per cent of the 1 cc. samples, after the water has been treated with chlorine.

Until the filtration plant is installed and in operation would it not be a good plan to apply an overdose of the chlorine to remove all B. coli. and then remove the excess of chlorine by the addition of sodium thio-sulphate applied at another location?

It is not the writer's intention to convey the idea that this water should not be filtered, but that more chlorine should be used to make the best of a doubtful case.

The estimated amount of sulphate of alumina required to treat the New York water (average 1 grain, maximum 2 grains per gallon) seems very generous.

At Harrisburg, Pa., where the length of the coagulation period is about the same as proposed for New York City, the average amount of sulphate of alumina used during the past year was 0.79 grains per gallon. The average turbidity was 56 with a maximum of 970 parts per million.

Of late it has been quite noticeable that about all expert commissions have advised mechanical filtration as the best and cheapest method of water purification.

The decision no doubt being due to the fact that this method will produce a water free from color and turbidity as well as from bacteria.

It is impossible to furnish a water free from color and turbidity with slow sand filters unless a coagulant is used, and this increases the cost of operation.

The delay in installing the New York City filtration plant seems to be a step backwards.

Mr. George A. Johnson (By letter): It was with great regret that the writer was obliged to be away from the city on the day of the first meeting of the New York Section of the American Water Works Association, for he wished to discuss Mr. Fuller's paper orally, and in other ways to further the interests of the new section so far as lay in his power to do.

The question of filtration of the Croton water supply, which Mr.

Fuller has covered so thoroughly, is one of immense importance to the welfare of New York and is one which seems to be but indifferently understood except by those who have come in immediate contact with it, and have followed it through the years that have passed. A great many people hold the view that the Croton water does not require filtration, and "point with pride" to the low typhoid fever rate as proof that it does not. It is true that New York City has one of the lowest typhoid fever death rates of any city in this country, but it is equally true that this death rate today is three times that of many large European cities. The difference in the typhoid death rates in New York City and in the European cities referred to means a loss to New York of some 300 lives per annum, and a modest estimate of the value of these lives, and the cost of treatment of the ten-fold number of cases where the patient recovers, represents a loss of vital capital to New York of some $2,000,000 annually in excess of that which would be lost were the typhoid death rate as low as it is in Berlin, Hamburg, Vienna, London and some other large European cities.

Those who have most to do with deciding whether or not New York shall have a filtered public water supply are men who know very little about the real character of the Croton water. In their city offices they have spring water, and in their homes they are supplied with either spring or filtered water. There is no first-class hotel in the city of New York which does not either supply spring water to its guests, or which does not maintain a filter. Consequently, not having to drink the water as drawn direct from the spigot, as the majority of the citizens of New York are obliged to do for reasons of economy, it is hard for them to realize that the water supply of this city is not the best in the world. The plain facts are that among all of the large cities of this country and western Europe, the water supply of New York at the present time is undoubtedly in many respects as bad as the worst, and very probably in some respects the worst of them all.

Of the physical defects of the Croton supply much could be said. Many of us know that during certain months of the year the water possesses an exceedingly offensive odor and taste, and is disagreeable to look upon. The writer lived for a number of years at the corner of 92d Street and Central Park West, and has no difficulty in recalling the fact that for weeks at a time the water as drawn from the spigot for drinking purposes and set away in the refrigerator to cool,

if allowed to stand over Sunday developed such an offensive taste and odor that it could not be so used. He also recalls the fact that the bad water was frequently covered with a green slime, and only the happy coincidence that he had a shower bath at hand prevented his emerging from the bath covered with this slime and smelling like a fish.

Ninety-second Street is immediately opposite the reservoir, and this body of water is used as a playground for seagulls. It is the custom of seagulls, as is well known, to obtain their food from the discharge of sewers around the water front. Is it not proper, therefore, to compare in some measure the habits of the seagull to those of the common house fly? In other words, is it not possible for the seagull to obtain infectious matter from the sewers along the water front, and carry such matter to and deposit it in the reservoir? The writer distinctly recalls the efforts of the watchman to prevent the daily aquatic sports of the seagull in the reservoir, in which endeavors he daily wastes a great deal of gunpowder. Since there is a fine of $25 for actually killing a seagull, it follows that the watchman uses no shot.

This last thought brings the writer back to conditions which exist in Bombay, India, where, on a nearby hill, are located the famous Towers of Silence, wherein are given to the vultures the bodies of the dead Parsees. The writer has viewed the ceremony of disposal of the dead in this way, and has observed the vultures flying away with long sections of intestines and other portions of the dead bodies. Apparently no exception is made of people dead of cholera or other infectious disease, all of them being disposed of in this same manner. Accordingly, it was necessary for the city officials of Bombay to cover all parts of their filter plant, and all of their distributing reservoirs, so that the water once it entered the purification plant should not be again exposed until drawn from the faucet; otherwise, if the filters and reservoirs were left open there was every possibility of gross contamination of the water being caused by the vultures dropping parts of infected bodies into it in their flight.

An important feature favoring the filtration of the Croton supply is the widespread use of spring water in New York City. In order to get some line on how extensive this industry is the speaker caused to have made a canvass of the offices in his office building. This canvass disclosed the fact that over one-half of the offices use spring water at a total annual cost of over $2000 a year. If we assume, as we well may, that all office buildings in New York follow this ex-

ample, and that many hotels, apartment houses and private residences do likewise, the annual cost of spring water consumed in the city of New York, largely because of the unsatisfactory quality of the existing public water supply, will run into the millions.

Some of the most prominent opponents of filtration of the Croton supply hold the view that the sterilization thereof with hypochlorites is a sufficient treatment. It is certainly strange, in view of the mass of literature bearing on this subject, which has appeared in recent years, that the advocates of this method of water purification have not learned that the hypochlorite treatment, while very valuable in emergencies for checking typhoid fever epidemics emanating from impure water, can only be considered in the final analysis as an adjunct to filtration. Particularly is this true where the water supply possesses marked physical defects, such as are characteristic of the Croton supply.

In the treatment of unfiltered water with hypochlorites it is necessary to use larger quantities of the germicide than when treating filtered water. In the endeavor to effect complete sterilization this often leads to the use of quantities of the chemical in excess of those which the water can absorb, and an exceedingly offensive taste and odor is imparted to the treated water. Not a few large cities have reversed their former judgment that the hypochlorite treatment would be sufficient, this decision in no small measure being due to difficulties actually experienced from tastes and odors of the chemical when used in the quantities considered necessary to produce adequate sterilization. Cleveland, Ohio, now designing a filtration plant, is one of these, and Jersey City, New Jersey, is rapidly approaching the day when mere sterilization of the impounded water supply with hypochlorites will prove unsatisfactory, and filtration be resorted to. At Trenton, New Jersey, where the relatively high typhoid fever death rate was markedly reduced by means of water sterilization with hypochlorites, a filter plant is nearing completion; and since the inauguration there of the hypochlorite treatment of the raw Delaware River water numerous well-founded complaints have been recorded because of the offensive taste imparted to the water when it became necessary to use large amounts of the chemical in order to sterilize the water when in its worst condition.

As early as 1910, in an address made before the Annual Convention of the American Public Health Association, held in Milwaukee, Wisconsin, the writer set forth in considerable detail certain limita-

tions of the hypochlorite treatment of water. His closing statements, all of which have been substantiated by actual experience in many cities during the past three years, were as follows:

In making a complete analysis of the practicability of this process, it is necessary to recognize the fact that it is not possible by the use of this germicide to overcome certain disadvantages, such as the following, which do not appear in connection with certain styles of water treatment:

1. Inability to remove or destroy all of the spore-forming bacteria not considered to be pathogenic to man, at least, not those common to water.
2. Inability to remove bacteria which are embedded in particles of suspended matter.
3. Inability to remove turbidity.
4. Inability to remove appreciable amounts of color or dissolved vegetable stain.
5. Inability appreciably to remove organic matter.
6. Inability to remove swamp tastes or odors.
7. Inability to remove creosote tastes or odors coming from the cleaning of stills used in the destructive distillation of wood.
8. Inability to soften water; as a matter of fact, the addition of hypochlorite of lime usually results in a slight increase in the hardness of the water, although this is not ordinarily measurable, notwithstanding the fact that the commercial product usually contains a little free quicklime which reduces slightly the carbonic acid in the water.
9. Difficulties encountered in applying this process, except with the greatest care, to waters which contain substantial quantities of reducing agents or compounds capable of oxidation, such as nitrites and unoxidized iron.

The foregoing statements set forth the advantages and shortcomings of this process which, like other things that are new, is likely to be, and in some cases is, considered a cure-all for all water ills, no doubt with disappointing results in some cases.

The application of hypochlorite to water, while comparatively simple, should always be carried out with much care and fidelity by a competent analyst; otherwise, if the dose is not adjusted so as to meet satisfactorily all local conditions, there is liable to be alternately an underdose of the chemical insufficient to sterilize or an overdose which will result in objectionable tastes and odors readily noticeable to the consumers, and due to the bleach itself.

The use of hypochlorites cannot be considered in the light of a substitute for filtration. Where waters are uniformly satisfactory in appearance, but open to suspicion as regards their content in bacteria, the use of the hypochlorite process alone in many cases may prove sufficient. Where waters are unsatisfactory in physical appearance and are also polluted and require filtration, the combined use of filters and the hypochlorite process is called for. As an adjunct to filtration processes it has a distinct field of applicability, as hereinbefore stated, for at a moderate cost it is feasible to obtain a water which is practically above suspicion; and, furthermore, there is brought about a substantial economy in the first cost of the filtration plant. This is made

possible by the use of higher rates of filtration than are ordinarily used, and the required filter area may therefore be reduced. It also effects a substantial economy in the cost of operation.

This is an age when people are demanding the best water supply obtainable. Twenty-five years ago only 300,000 people were supplied with filtered water in this country, while at this time over one-third of the urban population of the United States is supplied with water so purified. Public sentiment now demands that water supplies shall not only be free of disease germs, but that they also shall be clear, colorless, soft and free from objectionable tastes and odors. Almost all of the large inland cities have filter plants, or are building them. Of all the large Atlantic seaboard cities New York stands forth preëminently in the unenviable light of being too poor or too unwise to provide such a plant. For a city which controls 46 per cent of the commerce of the United States this assuredly is not a commendable position.

NOTE: This paper is open for discussion at the 34th annual convention, Philadelphia, Pennsylvania, May 11–15, 1914, and the discussion on it at that time, also any written discussion, will be printed in the next issue of the JOURNAL OF THE AMERICAN WATER WORKS ASSOCIATION.

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